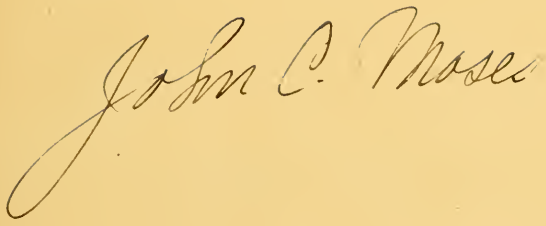
Class r620.6
Book No. 31421

Northeastern University
Library



TRANSACTIONS

OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS.

(INSTITUTED 1852.)

VOL XXXIV.

JULY TO DECEMBER, 1895.

NEW YORK:
PUBLISHED BY THE SOCIETY.

1895.

Entered according to Act of Congress, by the American Society of Civil Engineers, in the Office of the Librarian of Congress, at Washington.

Note.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

CONTENTS.

PAPERS.

NO.	MONTH.		PAGE
754	*July.*	Notes on Manufacture and Properties of Malleable Cast-Iron.	
		H. R. STANFORD	1
755	*do.*	The Water-Works of Syracuse, N. Y.	
		WILLIAM R. HILL	23
		Discussion on Paper No. 755.	
		By J. J. R. CROES	54, 57
		E. SHERMAN GOULD	56
		JOHN BOGART	58
		CHARLES E. EMERY	60
		J. N. GREENE	61
		L. L. BUCK	62
		G. L. CHRISTIAN	62
		Correspondence on Paper No. 755.	
		By J. L. VAN ORNUM	63
		JOHN W. HILL	63
		L. J. LE CONTE	64
		WILLIAM R. HILL	64
756	*August.*	The Temperature of Lakes.	
		DESMOND FITZGERALD	67
		Discussion on Paper No. 756.	
		By J. T. FANNING	110
		CHARLES G. DARRACH	110
		F. P. STEARNS	112
757	*do.*	The Philadelphia and Reading Terminal Railroad and Station in Philadelphia.	
		JOSEPH M. WILSON	115
		Discussion on Paper No. 757.	
		By CHARLES G. DARRACH	180
		Correspondence on Paper No. 757.	
		By SAMUEL TOBIAS WAGNER	181
		JOSEPH M. WILSON	184
758	*September.*	Consumption and Waste of Water.	
		DEXTER BRACKETT	185
		Discussion on Paper No. 758.	
		By DESMOND FITZGERALD	204
		L. FREDERICK RICE	204
		CHARLES G. DARRACH	204
		H. F. DUNHAM	204
		JOHN THOMSON	205
		J. J. R. CROES	207
		E. KUICHLING	208

3142/

NO.	MONTH.		PAGE
758	*September.*	Correspondence on Paper No. 758.	
		By L. J. Le Conte	211
		Clemens Herschel	212
		Robert Cartwright	213
		Daniel W. Mead	215
		Allen Hazen	218
		Dexter Brackett	221
759	*do.*	The Separate Sewer System Without Automatic Flush Tanks.	
		F. S. Odell	223
		Discussion on Paper No. 759.	
		By Charles G. Darrach	226
		Rudolph Hering	226
		Correspondence on Paper No. 759.	
		By Wynkoop Kiersted	227
		Kenneth Allen	228
		A. Prescott Folwell	230
		William B. Landreth	231
		L. J. Le Conte	232
		F. S. Odell	233
760	*do.*	Experiments on the Discharge of a 30-In. Stop Valve.	
		J. Waldo Smith	235
		Discussion on Paper No. 760.	
		By E. Kuichling	243
		Correspondence on Paper No. 760.	
		By J. Waldo Smith	246
761	*do.*	Construction of Substructure for Lonesome Valley Viaduct, Knoxville, Cumberland Gap and Louisville Railroad.	
		Gustave R. Tuska	247
		Discussion on Paper No. 761.	
		By George S. Morison	252
		Charles E. Emery	252
		Foster Crowell	252
		F. V. Abbot	253
		Bernard R. Green	253
		Correspondence on Paper No. 761.	
		By William Barclay Parsons	254
		George H. Pegram	255
		Walter H. Gahagan	256
		Gustave R. Tuska	258
762	*October.*	Topography on the Survey of the Mexico-United States Boundary.	
		J. L. Van Ornum	259
		Discussion on Paper No. 762.	
		By F. V. Abbot	279
		W. L. Webb	279
		Correspondence on Paper No. 762.	
		By Kenneth Allen	280
		William B. Landreth	281
		J. B. Johnson	283
		J. L. Van Ornum	284
763	*do.*	The Physical Qualities of Acid Open-Hearth Nickel Steel, as Compared with Carbon Steel of Similar Tensile Strength.	
		H. H. Campbell	285

NO.	MONTH.		PAGE
764	*October.*	What is the Life of an Iron Railroad Bridge?	
		J. E. GREINER	294
		Discussion on Paper No. 764.	
		By GEORGE S. MORISON	308
		T. C. CLARKE	309
		JOSEPH M. WILSON	309
		Correspondence on Paper No. 764.	
		By JAMES H. STANWOOD	310
		ONWARD BATES	312
		C. D. PURDON	314
		G. B. HAZLEHURST	315
		M. E. YEATMAN	317
		L. J. LE CONTE	318
		WALTER KATTE	318
		GEORGE H. PEGRAM	320
		R. MONTFORT	323
		J. P. SNOW	325
		THEOD. G. HOECH	327
		C. C. SCHNEIDER	328
		J. E. GREINER	335
765	*November.*	The Discharge of the Mississippi River.	
		WILLIAM STARLING	347
766	*December.*	Notes on High Masonry Dams.	
		JOHN D. VAN BUREN	493
		Discussion on Paper No. 766.	
		By WILLIAM E. WORTHEN	505
		GUSTAV LINDENTHAL	505
		ROBERT RIDGWAY	505
		F. W. SKINNER	505
		T. C. CLARKE	506
		EDWARD WEGMANN, JR	508
		Correspondence on Paper No. 766.	
		By KENNETH ALLEN	512
		JAMES DUANE	513
		C. M. BROOMALL	514
		M. S. PARKER	515
		L. J. LE CONTE	515
		H. F. DUNHAM	517
		JOHN D. VAN BUREN	517
767	*do.*	Hollow Tile Floors, Past and Present.	
		FR. VON EMPERGER	521
768	*do.*	Moving Two 36-In. Water Mains without Shutting Off the Water.	
		E. C. MOORE	532
		Discussion on Paper No. 768.	
		By EDWARD WEGMANN, JR	536
		JAMES OWEN	537
		JOSEPH C. PLATT	537
		Correspondence on Paper No. 768.	
		By DEXTER BRACKETT	538
		E. C. MOORE	540
769	*do.*	Tests of Fire-Proof Flooring Material.	
		GEORGE HILL	542

ILLUSTRATIONS.

PLATE.	MONTH.		PAPER	PAGE
I.	*August.*	Cellar Plan, Reading Terminal Station.....	757	131
II.	*do.*	Ground Floor Plan, Reading Terminal Station.	757	133
III.	*do.*	Galloway Boilers, Reading Terminal Station...	757	145
IV.	*do.*	Perspective Views of Reading Terminal Station during construction and completed.....	757	165
V.	*October.*	Views of Parts of Old Iron Bridges.....	764	313
VI.	*November.*	Discharge Curves at Columbus, Ohio, 1872.....	765	389
VII.	*December.*	View of 36-In. Pipe and Supports.....	768	535
VIII.	*do.*	View of Arch,—Testing Machine.....	769	545
IX.	*do.*	Views of Terra Cotta Arches.....	769	561
X.	*do.*	Views of Terra Cotta Arches.....	769	565

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

754.

Vol. XXXIV.—July, 1895.

NOTES ON MANUFACTURE AND PROPERTIES OF MALLEABLE CAST-IRON.

By H. R. STANFORD, Assoc. M. Am. Soc. C. E.

READ AT THE ANNUAL CONVENTION, JUNE, 1895.

If any systematic and scientific study has been made of malleable cast-iron, the results have been very carefully kept private. The reason for the apparent neglect may be that, until recently, only very unimportant shapes, as far as strength and uniformity were concerned, were made by the process, and specifications and inspections were not needed. The late extensive use of malleable cast-iron in the Master Car Builders' vertical plane coupler required the metal in a form in which it was subjected to very hard and uncertain usage, and where failure to withstand that usage might result in serious loss of life as well as property. The importance of quality in the casting led to the drop test being inserted in the Master Car Builders' specifications, and malleable cast-iron was first subjected to test treatment. In the hope of arousing ideas and discussions on this by no means simple and

unimportant industry, the following methods and opinions are submitted.

Malleable cast-iron is made principally from charcoal pig iron and a percentage of scrap or sprue sufficient to use the culled work, gates, runners and shrinkers that may result from the continued application of the process. After analyzing a number of kinds of pig iron, the author was led to believe that coke pig iron might just as well be used as charcoal pig, and developments have fully justified the assumption. The main difference between charcoal and coke pig iron is in the percentage of sulphur and in the cost; but if a good quality of coke is used, the sulphur is not high enough in the coke iron to affect its use. In Table No. 1 are results of chemical analyses and of physical tests of bars made from the different pig irons, and upon those results the use of charcoal iron was discontinued when the iron in stock was exhausted.

Sprue amounts to from 20% in heats for coupler or heavy work to 40 or 50% in light carriage work. An addition of 4% of heavy annealed scrap was introduced very successfully into all charges, and a large accumulation of the cheap and undesirable iron was disposed of. As much as 18% of annealed and broken couplers was tried experimentally in a charge for a coupler heat, and the resulting material was excellent, as far as quality was concerned, but the surfaces were not smooth and gave an impression of pock-marks. As annealed scrap can be secured at $4 a ton, and as a great many will take no exception to a rough-looking casting if of good quality, it seems as though there might be a use for the piles of broken couplers which a day's ride can hardly help revealing.

The furnaces used in melting malleable cast-iron are cupolas and open-hearth furnaces. The open-hearth furnaces which the author has used are of two types, a straight forced-draught furnace, using a good hard, close, bituminous coal as a fuel, with an air pressure of 4 or 5 ozs., and a Siemens-Martin furnace, using oil as the fuel. Fig. 1 shows sketches of the straight-draught furnace. In this furnace the fire was entirely drawn after the day's heats, and with no repairs could be used about three days. After three days the bottom needed a little fixing and the sides a little plaster, requiring an hour or two for the work. A renewal of some of the brickwork was necessary about every six months. A charge of 6 tons would be ready to tap three and one-quarter hours from the time it was charged. The consumption of fuel

depended on the number of tons a day melted, a certain amount of coal being necessary to heat the furnace for the first charge, whether 9 or 20 tons were melted. If 9 tons were melted, the coal consumed amounted to 0.6 lb. per pound of iron, and if 20 tons, to 0.45 lb. per pound. The furnace loss was about 4% of the charge. Two men were needed to operate the furnace, the melter and the fireman; and four additional men were needed for about 15 minutes for charging each 5-ton heat. The Siemens-Martin furnace, with oil as a fuel, was of the nature of an experiment. The furnace had a capacity of about 8 tons, and was equipped with three oil burners at each end. As operated practically, that is, fired hard for 12 hours, the consumption of oil was 450 galls. in 24 hours. The checkers were used only for heating the air which was used in addition to that for atomizing the oil. Air was found far superior to steam as an atomizing agent, and was supplied by a compressor working under an accumulator pressure of about 60 lbs. At times there was considerable water in the air as delivered at the burners, and to convert that water into superheated steam, rather than to heat the atomizing air, the air blast was conducted through a hot coil just before reaching the furnace. Oil, even when burnt with an excess of air, seems to decarburize the charge very little, differing in this respect very materially from producer gas. The advantage of the oil furnace was in the saving of fuel cost (which saving is, of course, a function of location of plant), in having no ashes or refuse to deal with, and in having a furnace which could be used to suit the convenience of the molders without a corresponding fuel and attendance loss. The great disadvantage, which might be overcome by additional experience, was that the checkers became clogged too soon, and too much time was required for cooling down, cleaning and reheating. The labor to operate amounted to about 25% more than was required to operate the coal furnace. The furnace loss amounted to 5 per cent.

The manipulation of the charge in the oil furnace was practically the same as in the coal furnace, and the times for melting and heating were also nearly the same. About one hour and a half was required to melt completely 6 tons, the solid masses being shifted to hurry the melting. After the charge was melted, it was frequently rabbled, and the surface skimmed clean of all slag after the first good rabbling. Just before tapping, a second skimming was made. Two tests were made before tapping a charge ; the first to see if the iron was "high enough,"

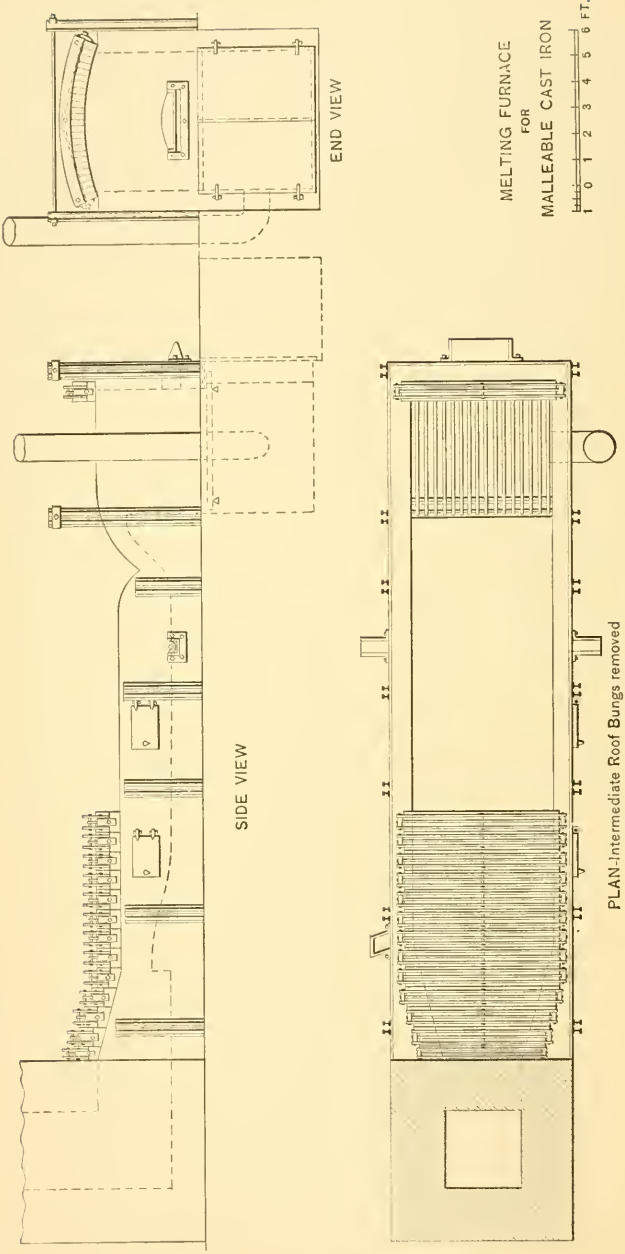

MELTING FURNACE
FOR
MALLEABLE CAST IRON
1 0 1 2 3 4 5 6 FT.
END VIEW
SIDE VIEW
PLAN-Intermediate Roof Bungs removed

Fig. 1.

that is, to see if in the heaviest form into which it was to be poured the carbon would be practically all in the combined state. The determination for a charge for ordinary heavy castings was made by dipping out a little of the iron and pouring a bar about 4 ins. long and 1 in. in diameter, and after it had set, cooling it carefully in water. If cooled too rapidly, the shrinkage was such that nothing could be told by the fracture, but if properly handled, in four minutes from the time of dipping out the iron the bar could be cast, cooled and broken. If the iron was "high enough," the fracture would be entirely white, but if not, there would be little spots of graphite about the size of the head of a small pin, and more or less numerous, depending upon whether the iron needed much more or little more time in the furnace. The second test was for fluidity and was made entirely by eye, using blue spectacles for protection. If thin enough, the iron was a clear white ; but if not it had a kind of dark hue not easily described. If the charge was intended for car couplers or other very heavy work, the test bar would be larger, or about $1\frac{3}{8}$ ins. in diameter. To secure thorough mixing in the furnace and to heat the hand ladles of the molders, the first ladleful drawn by each molder was returned to the furnace through a side trough ; but, notwithstanding the precaution, the mixing was very imperfect, as may be judged by a reference to the first form in Table No. 4.

The cupola was used very advantageously in connection with the open-hearth furnace. It was used for the lightest patterns and for patterns where a uniformly superior metal was not of vital importance and for shapes in which a considerable shrinkage would not destroy all strength. The lightest patterns were placed under the cupola because the fluidity of the cupola metal was greater than that of furnace metal, due to the higher percentage of carbon which resulted from the contact of the iron with the coke fuel. Ordinarily, if the charge is properly introduced, the quality of the product is very good, but at times the iron is not mixed in going down the cupola, and gray castings result. The high temperature of the iron and the excessive hardness which was necessary to avoid gray iron in the heavier shapes made the shrinkage of cupola iron destructive to some forms; as, for instance, a corner iron. In the cupola a much larger percentage of sprue could be used than in the furnace, because of the recarburizing tendency of the fuel. A high percentage of carbon in the iron is necessary for fluidity and clean castings, and on the hearth the bath loses rather

than gains carbon. Coke was used for fuel, and a small quantity of oyster shells or lime served to clean the iron and keep the cupola free from slag. About one-third of a pound of coke was used in melting 1 lb. of iron.

For small work hand ladles with a capacity of about 40 lbs. were used. Hard iron, if at the proper temperature for small work, scintillates brightly when tapped, differing from the more globular sputtering of gray iron.

Green sand, with a mixture of plumbago and cement for a facing, was used for molds. Most small shapes were cast with several pieces on a gate, and the pieces were so lightly attached to the runner that a slight blow was sufficient to detach them. Machines were advantageously used for making most molds.

Depending upon the form of the castings, they were next either tumbled or pickled. If the tumblers were filled properly it was surprising to see the delicate shapes that stood the treatment without breaking. The pickle was a very dilute sulphuric acid, which was poured over the castings, and after being allowed to stand for a short time, was removed, together with the loosened sand, by a water bath. The hard castings were then inspected, and the good ones chipped of fins and gate connections.

After chipping and sorting, the iron was taken to the packing-room where it was packed in pots for the annealing process. In order to avoid distortion or crooked work, and to get as much as possible in a pot, considerable care had to be exercised in the packing. Although the pots were jarred, and the packing consolidated as much as possible, there was always a further settling as the result of the heat in the annealing oven, and such a shape as a rim band for a buggy wheel, if placed with its axis horizontal, would be flattened to an elliptical form ; whereas, if packed with its axis vertical, the band would settle with the packing and would not be distorted. Clean, heavy forge scales seem to make the best packing, everything considered.

According to all accounts of the manufacture of malleable cast-iron which the author has been able to find, the packing must be an oxide of iron scale or some compound which, because of its chemical composition, with a certain degree of heat, can exercise a decarburizing influence upon the castings which it surrounds and so change the casting from a high carbon iron to a carbonless or wrought iron. The

author's opinion is that the packing need not be necessarily a decarburizing agent, but that it should be of such a physical form that it will readily fill all small interstices between castings; that it should not fuse or slag under the temperature needed for annealing; that it should not adhere strongly to the castings or form into hard lumps increasing the cost of dumping and of tempering the packing, and that it should not be too expensive. The analyses and tests given in Tables Nos. 2 and 3 are evidence that good malleable cast-iron is not a carbonless iron. An effort to anneal a brake shoe, so that the surface would be part hard and part soft, led to some specially prepared pots in which black oxide of manganese, clean river sand and sand mixed with a large proportion of ferro-cyanide of potassium were used respectively. In every case the iron was as soft and malleable as it could possibly have been by any treatment. A sprinkling of powdered quicklime, after the packing has been tumbled and raked, and moistened a little with a solution of sal ammoniac, makes it less liable to pack hard in the burning, and renders the dumping of the pots much easier. Riddling out the small particles of dirt and scale does not benefit the packing.

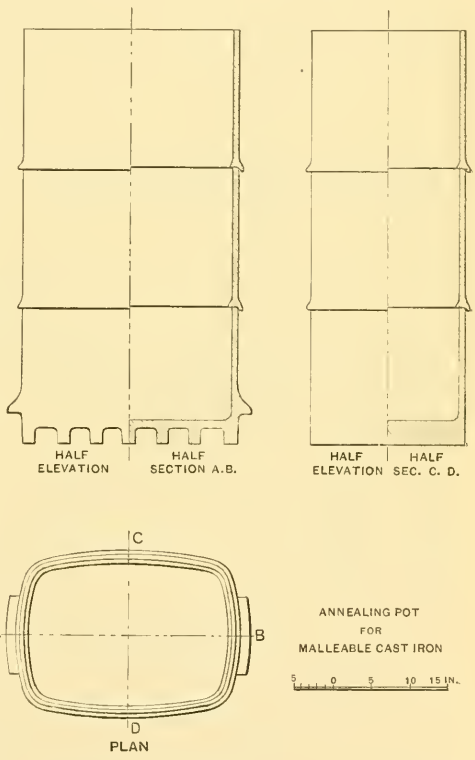


FIG. 2.

Pots were best made for convenience and economy in three sections, as seen in Fig. 2. A pot with a capacity of 800 lbs. of iron weighed 750 lbs., and lasted for about five heats of five days each. Each heat produced a heavy scale all over the outside of the pot, which

was easily knocked off after the pot had cooled, but limited its life. Before the pots were run into the oven, in order to keep the oven gases from entering them, the joints were plastered with a fire-clay mortar. To protect the pots against the waste caused by scaling, heavy coats of lime, of cement, of fire clay and of an asbestos cement were successively tried, but without success. The lime seemed to be the best protector, but the pots scaled under it so much as to make the labor of applying the whitewash an expense rather than a saving. A loaded pot weighed about 1 ton and was handled by six men with a carriage. Oil was superior to coal as a fuel for the annealing oven, not so much from the standpoint of economy as because of

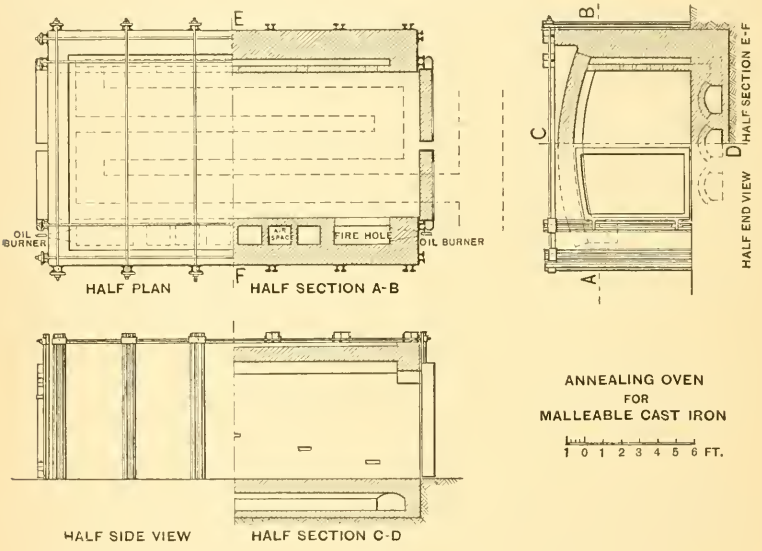


Fig. 3.

its controllability and cleanliness. Low-pressure air from a Root blower atomizes the oil satisfactorily, and 100 galls. of oil per burner per day was about the consumption. Fig. 3 shows sketches of an annealing oven. The temperature maintained was a bright cherry, and was determined by the eye through peep holes. Too high a temperature wasted the pots needlessly, burned the packing hard, and neither hastened nor improved the annealing of the iron. The time required to anneal iron varied more from the chemical composition than from the size or thickness of the piece, and ranged from 3 to 10 days, including the time necessary to raise the heat and cool the pots.

Combustion of the fuel in the annealing ovens should not be completed in the fire-hole, but by properly manipulating the chimney draft and the air supply, the mixed air and gas should slowly drift through the oven and combine as they come in contact with the red-hot pots. Uniformity of temperature throughout the oven is to be obtained only when the conditions are such that fuel gas is burned in all spaces in the enclosure.

As illustrating the above point, the effort may be cited to economize fuel in annealing by doing away with the murkiness which characterized the gases as they emerged from the fire-hole of an operating oven, but which, the projectors of the experiment failed to notice, nearly all disappeared before reaching the chimney inlet. The murkiness was rightly attributed to incomplete combustion in the fire-hole, and the argument at the base of the experiment was that if the air used for atomizing the oil was heated, instead of being injected into the oven cold, the resulting temperature in the fire-hole would be higher, and a larger part of the mixed fuel gas and air would combine and the amount of smoke in the issuing gases would be decreased, with a corresponding saving in fuel. By means of a coil placed in the chimney the atomizing air was heated to a temperature of several hundred degrees by means of the waste heat of the chimney. The device was introduced into one of two ovens, precisely alike in all other respects, and located side by side, loaded with the same kind of iron packed and placed in the same way. The oven heated with a smoky fire completed the work in four and one-half days, while in the other oven the oil valve had to be opened wider and six and one-half days were needed to anneal the iron farthest removed from the fire-hole, while the iron in the pots next to the fire-hole was badly burned.

The effect of composition on the time of annealing will be noticed later. A softer metal was obtained if the oven doors were allowed to remain closed for at least one day after the firing was stopped, and the sealing of the pots was much diminished. A day was sufficient to cool the pots with their contents to a black heat. Sometimes iron was packed in one large mass or bed, filling a part or the whole of the oven. The method effected a saving of pots, and cost no more in labor for packing, but was at a great disadvantage in time required to anneal the work thoroughly, in labor for unpacking and in the percentage of distorted pieces. A grade of iron requiring five days to anneal if packed

in pots would require seven days if packed in a bed. A per pound estimate of the productive cost of annealing iron, based on an oven heated by two oil burners and with a capacity of 30 pots, each loaded with 800 lbs. of iron, is as follows:

Labor packing pots	.0117	cent.
Labor loading oven	.0025	"
Fuel (including air for atomizing and attendance)	.0938	"
Labor unloading oven	.0025	"
Labor dumping pots and picking out the iron	.0104	"
Labor tempering and handling packing	.0083	"
Pots	.1000	"
Scale for packing	.0208	"
Total	.2500	"

After being annealed the freshly dumped iron has a rich blue color, which is probably given by a thin scale and is entirely lost in the tumbling that constitutes the last step in the process for making malleable cast-iron. Upon the use which is to be made of the iron depends the details of the final process. For shapes where appearance is of little importance, a rough tumbling for a short time is sufficient. For pieces such as handles and buggy trimmings, which are placed in conspicuous positions and are sometimes tinned or nickle-plated, a further treatment is necessary. After the rough tumbling follows a wet tumbling, in tumblers through which a current of water is running, which thoroughly cleanses the surfaces, and finally a dry polishing tumbling with pieces of leather.

Next to iron, the most important element in malleable cast-iron is carbon. A high percentage of carbon is necessary for fluidity; and fluidity is of prime importance, not only that the percentage of lost molds may be small, but that the iron may run clean and smooth, and the resulting work have a perfect surface. For strength and malleability the unannealed castings must have no graphitic carbon, but the total percentage must be in the combined state. With a given molten iron at a certain temperature, if the iron be poured into a mold of a certain section the casting will contain all of the carbon in the combined state, and the fracture of the annealed specimen will be of a uniform dark fibrous appearance. If the mold be of a smaller section, the casting made from the same iron will show the

total percentage of carbon as combined carbon, but there will be a surface chill, which in the annealed specimen will show as a white skin. If the mold be of a greater section, then the rate of cooling of the iron will be so slow that a part of the carbon will have time to separate from the iron as graphitic carbon, and will show in the fractured unannealed casting as scattered spots of graphite about as large as the head of a small pin, the number of spots increasing with the section of the casting. The above three castings, if annealed, will show the following physical properties. The first will have an ultimate strength of from 42 000 to 46 000 lbs., and will stretch and reduce about 6%, and may be considered a most desirable iron. The second will show in fracture a white crystalline skin, increasing in thickness as the section is lighter, with a black fibrous center; the ultimate strength will be about 52 000 lbs., and the stretch and reduction about 3.5%; if the section is so light that the whole fracture is white, then the iron will be very malleable and tough. The third casting will be little better than gray iron, with a strength of from 25 000 to 35 000 lbs. depending upon the percentage of graphite in the unannealed iron, and with practically no stretch or reduction.

In Tables Nos. 2, 3 and 5, all specimens, with the exception of bars No. 278 in Table No. 5, are examples of iron included between the limits of the first and second cases cited above. Bars No. 278 were cast from a cupola melting iron for a very light class of work, and samples of the iron which were tapped at the time the bars were poured, and were cast in light forms were of exceptionally good quality; the test bars, however, were of too heavy a section for the charge, and before being annealed showed a uniformly gray fracture. It is noticeable that the period of anneal does not seem to affect the strength of bars No. 278, but that they break erratically, a behavior explained probably by the dogs in the grip of the machine not bearing fairly on the specimen as introduced and necessitating a distortion of the bar which was beyond the strength of gray iron. From the nature of the material the test bars could not with justice be dressed. In the second form of Table No. 4 are given analyses of the skin and of the interior of a test bar showing the difference to be one of carbon. In the third form in the same table are given the physical properties of the interior of test bars of two different heats, the samples from the same heats differing only in time annealed; tests of untooled bars

corresponding with the above may be seen in Table No. 5. The thickness of the skin of the test bars averaged about $\frac{1}{16}$ in.

The chemical action in the furnace seems to be primarily the combining of the graphitic carbon of the charge with the iron, a combination made possible in the fused mass by the temperature; and secondarily a small burning out of carbon and slagging out of silicon and manganese. The secret of mixing is in using material containing combined and graphitic carbon in such proportions that at the temperature best adapted for pouring the graphitic carbon shall all have combined with the iron. To assist in preventing chilling in the mold, a limited amount of silicon is desirable. A chill is objectionable because in one piece there will be two different kinds of iron which have different physical properties and which do not act the same in service. Either kind of iron by itself would be stronger than in combination with the other. Chemical action involves atoms; therefore if all carbon exists in the combined state in the annealed iron, it must be distributed through the mass in infinitesimally small particles, and when these particles are liberated from the iron in the subsequent process of annealing, they must each be enclosed in an iron cell. Hence the definition might be given for malleable cast-iron that it is essentially a mixture of metallic iron and graphitic carbon, the carbon being in finely divided or atomic particles and the iron being the matrix for these particles. Gray iron differs from malleable cast-iron in that the carbon, instead of being in atomic particles, is in crystals, and these crystals cut the iron structure and make it discontinuous. In malleable cast-iron the continuous cellular iron structure is responsible for the strength of the product, and its malleability and ductility are limited by the non-deformable particles of graphite which occupy the cells. The process of making malleable cast-iron is then first to make the hard, brittle carbide of iron which is a stable compound at ordinary temperature. This step is possible because of the affinity of iron for carbon at high temperatures and the inability of the carbon to separate from the iron in the limited time required for the compound to cool in molds; and secondly, to change the carbon from the combined to the graphitic state by annealing. This step is possible because of the inability of the iron to hold carbon in combination if the compound is slowly cooled from a high to a low temperature.

Shrinkage is a function of casting temperature, and the nearer the iron is to the gray state and yet shows no graphite in fracture, the less will be the shrinkage. The normal shrinkage of hard iron is very nearly $\frac{1}{4}$ in. to the foot, and there is an expansion of about $\frac{1}{8}$ in. to the foot as the effect of the annealing process, so that the net shrinkage is about $\frac{1}{8}$ in. to the foot, or about the same as for gray iron. Inasmuch as shrinkage depends upon the condition of the carbon, and the condition of the carbon depends upon the section of the molds, and because abnormal shrinkage is so undesirable, the necessity arises for different mixtures for different patterns. The shrinkage of cupola iron prohibits its use for a great many shapes, because of the imperfect mixing that necessarily characterizes cupola melting, and because of the factor of safety required in the charge to be sure of avoiding gray iron in the castings.

After carbon, sulphur is the next important element in malleable cast-iron. Sulphur tends to hold the carbon in combination with the iron and gives a stronger product because of the semi-steel which it produces. Sulphur is undesirable because of the hindrance it offers to annealing. The shapes ordinarily made by this process need not have great strength, but it is desirable that they be soft and capable of bending, and that the time of manufacture and cost be as low as possible. The effect of sulphur upon strength is well illustrated in Table No. 1. To show how sulphur affects the time necessary to anneal, couplers which analyzed about 0.040% in sulphur, and in which were sections about $1\frac{1}{2}$ ins. thick, were thoroughly annealed in $3\frac{1}{2}$ days, while iron bands for buggy wheels, which were no more than $\frac{3}{16}$ in. thick and analyzed about 0.150% in sulphur, were invariably hard if given less than five days. If sulphur is carried as high as 0.200%, enough carbon is retained in the combined state to give to fractures a uniform crystalline appearance, and the method is employed to make a so-called hard or special steel. This product in ordinary sections does not anneal in less than nine days. Special pains should be taken when buying coke for cupola melting to get a coke low in sulphur, as the iron coming in contact with the fuel picks up sulphur, and delay in the annealing process, with the accompanying wastes, is the result.

The bars used for making the physical tests given in this paper were of two different kinds. For all heats bearing a number less than

361, the test bars were cast rectangular in section, 2 ins. long between shoulders, and approximately $\frac{1}{2}$ in. thick and $\frac{3}{4}$ in. wide, and as shown in Fig 4.

It being very unusual to machine any form made of malleable cast-iron, it would have been unfair to prepare the test bars by filing or dressing, as such treatment would have removed the strongest portion, the skin; and a tool, no matter how sharp, tends to drag the metal and so leave small scratches or furrows, which offer starting places for breaking. An undressed cast form is necessarily irregular, and this irregularity, increased by the draft of the pattern, made the measurement of the original area and of the fractured area rather unsatisfactory work, and that the error might be partially eliminated, several bars of each kind were broken and the results averaged. With the idea that a cylinder can be cast more perfectly than a rectangle and that in a cylinder the shrinkage is not so damaging as in a rectangle, the form of the test bar was changed, and, beginning with specimen

Fig. 4.

No. 361, a bar 2 ins. between shoulders and $\frac{13}{16}$ in. in diameter was used. In the first set of bars the unannealed sample was obtained from the part marked *A* in the sketch, and was taken from the same bar, which was afterwards drilled in the flat part for the annealed sample. For the cylindrical specimens, one of the unannealed bars was broken and the sample taken from the portion $\frac{13}{16}$ in. in diameter, and the annealed specimen was taken from near the fracture of one of the tested bars. In every case more weight should be given to elongation than to reduction of area, because of difficulties in determining the exact areas for computing the latter. With the exception of one case, specially tabulated in Table No. 2, and the heats tabulated in Table No. 5, the bars composing the sets were cast from the same ladleful of iron, drawn about the middle ladleful of the heat. All test bars received the same treatment in the annealing oven as far as position in the annealing pots and location in the oven were concerned.

The results obtained are not sufficiently numerous and uniform to warrant any general conclusions regarding the effect of manganese and phosphorus further than to say that phosphorus seems to be a very passive element and anything but the bugbear which it is in steel. A

limited period of anneal, judging from the results in Table No. 5, might be considered as giving better results than an indefinitely longer one, both as regards strength and malleability. The fact that there was no sure way of telling whether bars were thoroughly annealed and yet not weakened by over anneal makes the physical results given a little uncertain as representing an iron corresponding to the analyses accompanying the results, and only by averages of many tests can the truth be known.

The chemical composition of the specimens in Table No. 5 is not given, as at the time those bars were cast the importance of pouring all the bars in one set from one ladleful, in order to have them of the same composition, was not appreciated. As there were so many bars in those sets, several ladlefuls of iron were required to pour them, and being drawn by one man they differed widely, and, therefore, the analyses are not reliable. Carbon, however, seems to be little affected, either in total percentage or combined carbon, and the other impurities are not affected at all.

The difference in the average percentage of loss of carbon in Tables Nos. 2 and 3 may reasonably be attributed to the difference in the form of the test bars in the two tables. In Table No. 2 the bars were rectangular and the average loss of carbon is over 50% more than for the cylindrical bars in Table No. 3. To get the drillings a $\frac{5}{16}$-in. drill was used and was entered into the bar in both cases till it became fouled in the skin on the other side of the bar. Allowing the skin to be $\frac{1}{16}$ in. thick in the rectangular bar, $\frac{6}{16}$ in. of center was added to the drilled sample and $\frac{1}{16}$ in. of skin, while in the cylindrical bars $\frac{11}{16}$ in. of center was taken and $\frac{1}{16}$ in. of skin. From the second form in Table No. 4 the skin is seen to be much lower in total carbon and higher in combined carbon than the interior, and making allowance for the difference, the real loss of carbon for the two tables is found to be nearly the same.

The chemical results presented in this paper were obtained by methods elaborated in the work by Blair on the "Chemical Analysis of Iron" and in general were as follows: The total carbon was determined by treating the sample with the double chloride of copper and potassium, filtering after the copper was all dissolved, burning the dried residue in a current of oxygen and collecting the carbon as carbon dioxide in a solution of potassium hydrate. For graphitic carbon the

sample was treated with nitric acid, filtered, and the dried residue treated as in analyzing for total carbon. The combined carbon was obtained by taking the difference between the results obtained for total carbon and for graphitic carbon. Manganese was determined by treating the sample with nitric acid, then boiling with additional nitric acid and peroxide of lead, and after cooling determining the percentage of manganese by color comparison with a standard treated in the same way and at the same time. To determine silicon the sample was treated with nitric acid, and after all action had ceased sulphuric acid was added and the mixture boiled until fumes of sulphur trioxide were given off. It was then cooled, diluted, heated until the sulphate of iron was entirely dissolved, filtered hot on an ashless filter, ignited, and the crucible with its contents weighed; after treating the contents of the crucible with sulphuric and hydrofluoric acids, evaporating to dryness and igniting again, the crucible with its contents was weighed and the difference between the two weights considered silica. Phosphorus was obtained from the weighed amount of phospho-molybdate precipitated from a nitric acid solution of the sample on the addition of an ammoniacal solution of molybdic acid. The percentage of sulphur was found by treating the sample with nitric acid, and after all action was complete a little sodium carbonate added and the solution evaporated to dryness; after cooling hydrochloric acid was added and the solution evaporated to dryness; redissolved in as little hydrochloric acid as possible, the solution was diluted and filtered; the addition of barium chloride to the boiling filtrate gave a precipitate of barium sulphate in which the sulphur could be calculated.

The author acknowledges obligation to the New Jersey Steel and Iron Company of Trenton, N. J., and to Prof. J. B. Johnson of Washington University, St. Louis, Mo., for the use of testing machines for making the physical tests recorded in the accompanying tables.

TABLE No. 1.

COMPARING PROPERTIES OF MALLEABLE CAST-IRON MADE FROM CHARCOAL PIG IRON AND FROM COKE PIG IRON.

From Charcoal Pig.					From Coke Pig.				
Heat number.	Number of bars averaged.	Maximum strength per square inch.	Per cent. reduction.	Per cent. elongation.	Heat number.	Number of bars averaged.	Maximum strength per square inch.	Per cent. reduction.	Per cent. elongation.
3 ..	3	46 500	4.5	3.2	153..	3	46 100	4.0	5.2
4...	2	42 300	4.1	2.8	166 A	3	51 000	4.2	6.8
7...	3	46 100	6.2	3.5	177..	4	49 100	8.1	9.2
9...	3	41 200	2.2	4.5	194..	2	48 800	3.7	4.0
10...	3	41 500	1.7	4.6	199..	1	45 800	4.1	5.0
13...	3	40 800	4.4	3.0	275..	3	43 100	2.1	5.0
14...	3	40 300	3.5	2.7	276..	4	41 100	2.8	5.1
15...	2	40 400	3.8	4.0	282..	4	49 500	5.4	4.9
16. .	3	39 400	3.8	4.2	314..	3	42 000	3.7	5.5
17...	3	40 600	5.2	4.2	315..	4	43 500	3.6	6.4
18...	3	42 100	4.8	4.0	361..	3	56 100	2.6	5.7
22...	2	44 800	2.6	4.0	362 .	3	52 000	6.6	8.0
110...	3	46 400	5.1	3.8	391..	3	55 100	5.5	5.2
113...	3	46 200	3.9	4.5	459..	3	44 200	2.1	4.2
117...	6	45 000	3.7	4.2	469..	3	51 600	7.7	7.0
Average.....		42 910	3.81	3.97	Average.....		47 930	4.41	5.81

NOTE.—Charcoal pig mixes averaged about 10% coke pig iron. Coke pig mixes averaged about 10% charcoal pig iron. Sulphur in first set of bars averages 0.042. Sulphur in second set of bars averages 0.063.

TABLE No. 2.

Showing Chemical Composition (unannealed and annealed) and Physical Properties of Malleable Cast-Iron.

Heat No.	Unannealed or annealed.	Total carbon.	C. C.	G. C.	Mn.	Si.	P.	S.	Loss of carbon.	Maximum strength per square inch.	Per cent reduction.	Per cent. elongation.	Number of bars averaged.	Hours annealed.
44.....	Sprue.....	3.01	2.84	0.17	0.25	0.47	0.236	0 041	0.29	50 800	5.0	5.5	1	120
	Annealed.....	2.72	0.32	2.40	0 26	0.47	0.237	0.043						
49.....	Sprue.....	3.15	2.49	0.66	0.28	0.50	0.230	0.029	0.29	54 000	2.8	5.0	3	120
	Annealed.....	2.86	0.12	2.74	0.29	0.50	0.223	0.030						
50.....	Sprue.....	3.25	3.02	0.23	0.21	0.51	0.169	0.034	0.51	50 000	4.0	4.7	3	120
	Annealed.....	2.74	0.41	2.33	0.20	0.51	0.174	0.033						
65.....	Sprue.....	3.73	3.63	0.10	0.24	0.66	0.187	0.047	1.31	46 800	4.0	4.2	3	120
	Annealed.....	2.42	0.07	2.35	0.24	0.64	0 191	0.047						
67.....	Sprue.....	3.64	3.22	0.42	0.28	0.63	0.212	0.054	1.12	44 900	4.0	5.0	2	120
	Annealed.....	2.52	0.94	1.58	0.27	0.65	0.210	0.0 2						
91.....	Sprue.....	2.86	2.73	0.13	0.18	0.59	0.171	0.044	0.42	48 600	6.8	7.4	4	120
	Annealed	2.44	0.05	2.39	0.18	0.58	0.172	0.044						
92.	Sprue.....	2.98	2.82	0.16	0.17	0.51	0.171	0.052	0.76	52 000	7.2	7.4	4	120
	Annealed.....	2.22	0.38	1.84	0.16	0.51	0.171	0.052						
97.....	Sprue.....	3.15	2.92	0.23	0.18	0.48	0.148	0.046	0.30	45 900	3.1	4.8	3	120
	Annealed.....	2.85	0.13	2.72	0.19	0.46	0.145	0.042						
98.....	Sprue.....	2.94	2.60	0.34	0.26	0.65	0.137	0.050	0.70	49 700	6.2	7.7	3	120
	Annealed.....	2.24	0.20	2.04	0.26	0 61	0.135	0 052						
138.....	Sprue.....	2.90	2.79	0.11	0.31	0.99	0.192	0.044	0.91	47 800	1.4	3.8	3	120
	Annealed.....	1.99	0.10	1.89	0.32	0.97	0.191	0.042						
140.....	Sprue.....	2.79	2.62	0.17	0.33	0.87	0.194	0.046	0.50	54 900	4.0	5.2	3	120
	Annealed.....	2.29	0.38	1.91	0.34	0.88	0.194	0.045						
141.....	Sprue.....	2.80	2.56	0.24	0.31	0.91	0.196	0.035	0.53	50 800	3.7	5.2	4	120
	Annealed.....	2.27	0.15	2.12	0.31	0.93	0.200	0.036						
166.....	Sprue.....	3.24	3.21	0.03	0.31	0.44	0.152	0.055	0.82	48 900	4.1	4.3	3	120
	Annealed.....	2.42	0.12	2.30	0 30	0.44	0.154	0.056						
194.....	Sprue.....	3.30	3.23	0.07	0.35	0.77	0.159	0.022	0.78	48 800	3.7	4.0	2	120
	Annealed.....	2.52	0.06	2.46	0.32	0.79	0.169	0.0:2						
282.....	Sprue.....	3.31	3.18	0.13	0.21	0.62	0.153	0.097	1.06	42 300	3.2	4.1	4	108
	Annealed.....	2.25	0.08	2.17	0.22	0.64	0.151	0.096						
Av.....	Sprue.....	3.14	2.93	0.21	0.26	0.64	0.181	0.046	0.69	49 100	4.2	5.2	3	119
	Annealed	2.45	0.23	2.22	0.26	0.64	0.181	0.046						

Note.—The above test bars were all rectangular in section.

TABLE No. 3.

SHOWING CHEMICAL COMPOSITION (UNANNEALED AND ANNEALED) AND PHYSICAL PROPERTIES OF MALLEABLE CAST-IRON.

Heat No.	Unannealed or annealed.	Total carbon.	C. C.	G. C.	Mn.	Si.	P.	S.	Loss of carbon.	Maximum strength per square inch.	Per cent. reduction.	Per cent. elongation.	Number of bars. averaged.	Hours annealed.
391..	Sprue	3.02	2.96	0.06	0.18	0.69	0.138	0.066	0.07	55 100	5.5	5.2	3	108
	Annealed	2.95	0.15	2.80	0.19	0.69	0.140	0.064						
395..	Sprue	3.09	2.99	0.10	0.19	0 60	0.133	0.060	0.11	43 800	6.1	6.3	3	108
	Annealed	2.98	0.53	2.45	0.19	0.62	0.130	0.061						
400..	Sprue	3.09	2.96	0.13	0.18	0.75	0.142	0.065	0.53	44 900	7.7	10.3	3	108
	Annealed	2.56	0.31	2.25	0.19	0.74	0.142	0.061						
406..	Sprue	3.07	2.77	0.30	0.19	0.65	0.163	0.064	0.16	47 000	7.2	6 2	3	108
	Annealed	2 91	0.08	2.83	0.20	0.64	0.164	0.064						
422..	Sprue	3.26	2.65	0.61	0.17	0.69	0.156	0.071	0.31	45 300	9.1	8.2	3	108
	Annealed	2 95	0.36	2.59	0.18	0.61	0.151	0.068						
433..	Sprue	2.85	2.72	0.13	0.18	0.74	0.161	0 073	0.08	64 500	1.3	2.8	3	108
	Annealed	2.77	0.72	2.05	0.18	0.72	0.162	0.071						
436..	Sprue	2.88	2.73	0.15	0.18	0.90	0.196	0.069	0.30	38 900	4.3	6.2	3	108
	Annealed	2.58	0.51	2.07	0.18	0.87	0.192	0.069						
445..	Sprue	2.97	2.75	0.22	0.18	0.77	0.148	0.073	0.31	69 100	2.6	4.0	2	108
	Annealed	2.66	0.31	2.35	0.19	0.76	0.145	0.075						
451..	Sprue	3.08	2.85	0.23	0.18	0.96	0.123	0.033	0.93	45 400	7.2	8.0	2	108
	Annealed	2 15	0.08	2.07	0·19	0.96	0.129	0.039						
458..	Sprue	3.08	2.82	0.26	0.18	0.74	0.151	0.036	1.05	56 700	8.4	8.2	2	108
	Annealed	2.03	0.28	1.75	0.18	0.71	0.150	0.037						
459..	Sprue	3.03	2.81	0 22	0.19	0.70	0.195	0.038	0.97	44 200	2.1	4.2	3	103
	Annealed	2.06	0.27	1.79	0.20	0.70	0.192	0.037						
469..	Sprue	3.09	3.00	0.09	0.20	0.77	0.127	0.022	0.23	51 600	7.7	7.0	3	108
	Annealed	2.86	0.32	2.54	0.19	0.75	0.129	0.023						
474..	Sprue	3.08	2.92	0.16	0.23	0.70	0.168	0.023	0.41	46 600	9.8	8.5	3	108
	Annealed	2.67	0.09	2.58	0.22	0.70	0.156	0.023						
495..	Sprue	2.84	2.62	0.22	0.31	0.63	0.182	0.014	0 26	48 100	8.6	8.7	3	108
	Annealed	2.58	0.06	2.52	0.32	0.66	0.178	0.018						
510.	Sprue	3.26	3.23	0.03	0.40	0.64	0.136	0.040	0.14	46 000	5.9	5.3	3	108
	Annealed	3.12	0.53	2.59	0.40	0.67	0.135	0.039						
Av..	Sprue	3.04	2.85	0.19	0.21	0.73	0.154	0.050	0.38	49 810	6.23	6.61	2.8	108
	Annealed	2.66	0.31	2.35	0.21	0.72	0.153	0.050						

NOTE.—The above test bars were all cylindrical in section.

TABLE No. 4.

SHOWING VARIATION IN CHEMICAL COMPOSITION AND PHYSICAL PROPERTIES OF IRON TAPPED FROM THE SAME HEAT BUT AT DIFFERENT TIMES.

Heat.	Unannealed or annealed.	C. C.	G. C.	Mn.	Si.	P.	S.	Maximum strength per square inch.	Per cent. reduction.	Per cent. elongation.	Hours annealed.	REMARKS.
282...	Sprue.....	2.98	0.18	0.21	0.88	0.152	0.091	40 200	3.8	4.0	108	Poured from first ladle drawn from heat.
								39 800	.....	.....	"	
	Annealed	0.38	2.41	0.21	0.80	0.156	0.091	39 400	3.4	4.0	"	
								38 700	4.8	4.5	"	
	Sprue.....	3.18	0.13	0.21	0.62	0.153	0.097	42 100	4.4	4.0	"	Poured from middle ladle drawn from heat.
								42 700	2.9	5.0	"	
	Annealed	0.08	2.17	0.22	0.64	0.151	0.096	42 100	3.1	4.0	"	
								42 400	2.2	3.5	"	
	Sprue.....	2.64	0.15	0.22	0.68	0.173	0.099	47 600	5.8	5.0	"	Poured from last ladle drawn from heat.
								50 700	4.5	5.5	"	
	Annealed	0.59	1.96	0.22	0.66	0.165	0.100	47 800	5.5	3.5	"	
								51 900	5.7	5.5	"	

NOTE.—Second specimen in table broke in shoulder.

CHEMICAL COMPOSITION OF SKIN AND OF INTERIOR OF AN ANNEALED SPECIMEN.

Heat No.	Sample taken from	C. C.	G. C.	Mn.	Si.	P.	S.	Hours annealed.
361	Skin	0.61	0.73	0.15	0.65	0.154	0.116	120
	Interior.....	0.02	3.49	0.14	0.63	0.158	0.113	120

TABLE No. 4—(Continued).

PHYSICAL PROPERTIES OF SPECIMENS FROM WHICH ALL THE SKIN HAS BEEN REMOVED.

Heat No.	Days annealed.	Original diameter of test bar.	Reduced diameter of test bar.	Maximum strength per square inch.	Per cent. reduction.	Per cent. elongation.	REMARKS.
		Inches.	Inches.				
361	5	0.800	.648	44 200	.....	.....	Broke in shoulder.
361	9	0.800	.643	41 500	1.2	2.5	
362	5	0.800	.646	39 500	4.3	4.5	
362	7	0.800	.657	41 200	2.1	3.0	
362	9	0.800	.649	37 600	3.3	3.5	

NOTE.—The properties of undressed bars from the above heats may be found in Table No. 5.

TABLE No. 5.

Showing Varying Physical Properties of Malleable Cast-Iron Test Bars Depending Upon Time Annealed.

Heat number.	Maximum strength per square inch.	Per cent. reduction	Per cent. elongation.	Hours annealed.	Heat number.	Maximum strength per square inch.	Per cent. reduction.	Per cent. elongation.	Hours annealed.
	42 000	4.1	6.0	79		46 800	3.9	5.0	120
	39 500	4.2	5.5	79		43 100	4.0	5.0	120
	37 200	3.9	5.0	79		44 500	.....	.....	120
	41 800	.....	8.0	79		45 300	5.2	5.0	168
	39 600	4.3	5.0	108	67....	42 800	3.4	3.5	168
	40 800	2.4	5.0	108		44 300	4.3	4.5	168
	41 800	3.5	6.0	108		48 900	5.3	5.0	216
	42 100	1.0	4.5	108		47 200	7.0	5.5	216
276.....	40 200	7.5	8.0	192		48 600	6.7	5.0	216
	42 800	8.2	8.0	192		18 600	0.0	0.0	79
	42 300	7.9	9.0	192		21 800	0.0	0.0	79
	43 800	10.7	8.5	192		11 500	0.0	0.0	79
	39 800	7.9	8.0	216		12 100	0.0	0.0	79
	40 500	4.7	6.5	216		18 000	0.0	0.0	108
	38 100	6.6	7.0	216	278....	18 100	0.0	0.0	108
	39 800	2.9	6.5	216		26 300	0.0	0.0	108
	44 200	7.2	5.5	72		23 700	0.0	0.0	108
	43 200	5.1	5.0	72		11 400	.....	.....	192
	42 900	4.9	4.5	72		10 200	0.0	0.0	192
	50 400	3.0	5.5	120		6 300	0.0	0.0	216
	49 100	3.5	5.0	120		56 900	0.8	4.0	120
	50 600	4.8	4.5	120		55 400	4.2	5.0	120
	48 800	4.3	5.0	120		56 100	2.7	8.0	120
101.....	46 900	4.6	6.0	192		59 600	5.3	4.0	168
	47 700	3.5	5.5	192	361....	58 100	3.5	5.0	168
	46 000	6.1	4.5	192		52 100	1.6	2.0	168
	44 500	5.7	5.5	192		54 800	5.1	5.0	216
	47 400	8.6	7.0	216		52 800	4.1	3.5	216
	47 300	12.6	7.0	216		50 300	3.7	4.5	216
	47 300	5.3	6.5	216		50 700	5.9	8.0	120
	40 400	5.7	7.0	79		52 600	6.5	7.5	120
	42 300	5.9	7.5	79		52 700	7.3	8.5	120
	42 800	2.8	6.5	79		48 100	5.7	6.0	168
	44 300	2.4	5.5	108	362....	58 000	4.7	4.0	168
	44 600	1.7	5.0	108		47 200	5.3	6.0	168
	43 500	2.2	4.5	108		46 700	5.1	5.0	216
275.....	42 100	6.0	7.5	192		47 500	3.8	4.0	216
	40 000	9.0	6.5	192		51 100	3.6	5.0	216
	38 800	8.3	6.0	192					
	42 100	4.6	7.5	192					
	41 200	6.4	8.5	216					
	40 600	7.3	6.5	216					
	36 300	6.0	5.0	216					
	42 200	8.4	8.5	216					

Note.—Where the percentage of reduction or elongation is not given, the specimen broke in the shoulder.

Bars 278 were poured from a cupola heat charged for light work, and showed gray in fracture before annealing.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

755.

Vol. XXXV.—July, 1895.

THE WATER-WORKS OF SYRACUSE, N. Y.

By WILLIAM R. HILL, M. Am. Soc. C. E.

READ MAY 15TH, 1895.

WITH DISCUSSION.

On June 21st, 1888, the Hon. Wm. B. Kirk, Mayor of the city of Syracuse, N. Y., appointed a commission to select the best available source for furnishing an abundant supply of water for public, mechanical and domestic uses. At that time a small portion of the city was supplied with water from Onondaga Creek by the Syracuse Water Company, and with spring water by parties who sold it by the gallon, delivering it in bottles and from tanks. Rain water was collected on roofs and stored in cisterns for laundry and other purposes.

The Commissioners engaged J. J. R. Croes, M. Am. Soc. C. E., to take charge of the investigation, and the author was appointed his principal assistant. After a thorough investigation of eleven different sources, Mr. Croes recommended "the water of Skaneateles Lake to be so far superior to all the other waters examined that it should be obtained for Syracuse if it could possibly be secured."

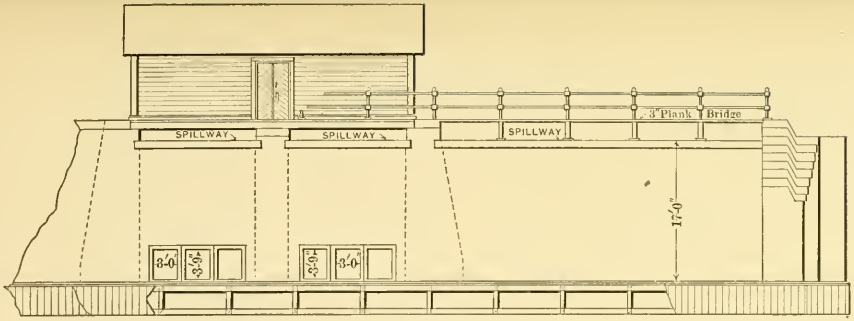
Skaneateles Lake is situated about 19 miles southwesterly from Syracuse and at an elevation of 466.40 ft. above the Erie Canal at Syracuse, or 867.096 ft. above mean low tide at Albany, N. Y. The lake is 15 miles long and generally about 1 mile wide, with an area of $12\frac{3}{4}$ square miles. Its greatest depth is said to be about 350 ft. Its shores are bold and free from vegetable growth. Its water-shed is steep and narrow, with an area (exclusive of the lake) of 60.28 square miles, the crest line of which ranges generally from 500 to 1 100 ft. above the surface of the lake.

The following table, taken from Mr. Croes' report, gives the results in parts per 100 000 of the chemical analyses of Skaneateles Lake water as made by Professor H. H. Cornwall, Charles Harrington, M.D., and Professor Elwyn Waller:

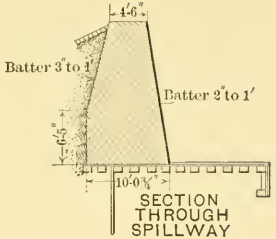
	By Cornwall, September 7th, 1888.	By Harrington, November 12th, 1888.	By Waller, November 23d, 1888.	Average.
Solids, volatile	0.300	2.800	1.700	1.600
" fixed	12.200	11.800	10.300	11.433
Total solids	12.500	14.600	12.000	13.033
Hardness, temporary	6.40	.....	9.50	7.95
" permanent	2.90	.....	0.71	1.80
Total hardness	9.30	8.00	10.21	9.17
Sulphates	....	....	0.497	0.497
Nitrates	0 000	0.000	0.068	0.023
Chlorine	0.430	0.310	0.319	0.353
Free ammonia	0.0040	0.0034	0.0030	0.0035
Albuminoid ammonia	0.0080	0.0118	0.0045	0.0080
Oxygen consumed	0.06	.....	.....	0.06

In December, 1888, biological examinations were made by Charles G. Currier, M.D., of New York City. In 1 cu. cm. of Skaneateles Lake water he found 21 living bacteria; in Onondaga Creek water, 491; and in the Croton water supplied to New York City, 673. He reports, "although a large number of living bacteria need not in itself give sufficient ground for pronouncing a water objectionable, still, the fewer bacteria present, the nearer is the water to the ideal of excellence," and that the bacteria in Skaneateles Lake are "of an entirely harmless character."

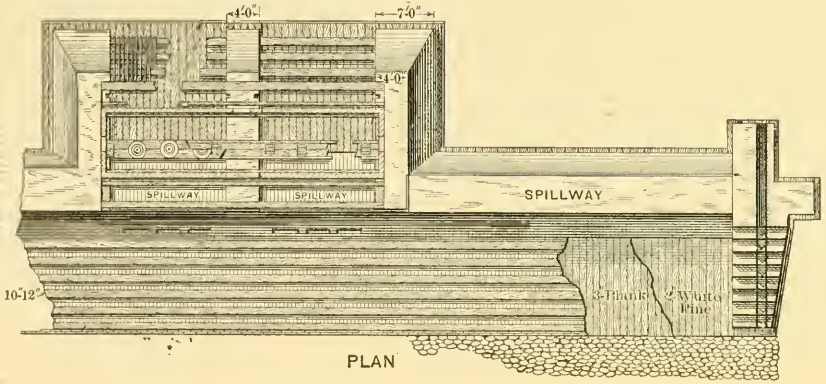
In June, 1889, the citizens of Syracuse, at a special election, declared almost unanimously in favor of Skaneateles Lake water and municipal ownership. A water board composed of six members, three



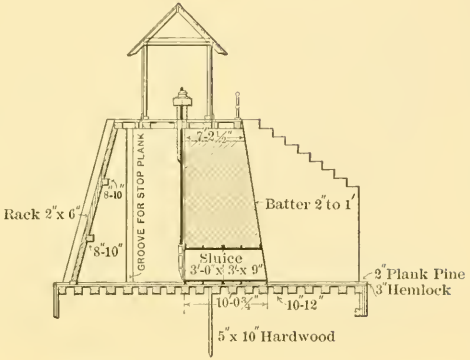
ELEVATION



SECTION THROUGH SPILLWAY



PLAN



SECTION THROUGH BULKHEAD

FIG. 1.

from each of the two principal political parties, was then appointed by the Mayor, as authorized by Chapter 291 of the Laws of 1889. This board was "authorized to acquire, construct, maintain, control and operate a system of works, with water from Skaneateles Lake." The author was appointed chief engineer, and associated with him as consulting engineer was Howard Soule, M. Am. Soc. C. E., to whom the author is much indebted for invaluable aid and counsel.

The old water-works were owned and operated by the Syracuse Water Company, and consisted of a pumping station, three reservoirs, and 39⅔ miles of street mains, 70% of which were 4-in. and 6-in. pipes. The main supply was pumped from Onondaga Creek into a distributing reservoir, this supply being slightly increased by water from springs flowing directly into the reservoir. The flow line of the reservoir is at an elevation of 108 ft. above the canal. The pumping plant consisted of one Worthington compound duplex condensing engine, with a capacity of 10 000 000 galls. daily, and two Dean compound duplex condensing engines, each with a capacity of 3 000 000 galls. daily.

The city then had a population of about 90 000, with 172 miles of streets and only 39⅔ miles of water mains. Furthermore, the water furnished was quite unfit for either domestic or manufacturing purposes, and afforded a very inefficient fire protection.

Skaneateles Lake is a feeder of the Jordan level of the Erie Canal, this supply being controlled by sluice gates in the state dam at the foot of the lake. On the outlet between the lake and the canal, a distance of 9 miles, there are 25 dams, furnishing head to wheels with a capacity aggregating about 3 000 H. P.

In addition and preliminary to the new works proper, it was necessary to acquire the water power rights on Skaneateles Creek, also the plant of the Syracuse Water Company, and to meet the requirements in the law authorizing the city to take water from Skaneateles Lake, it was necessary to increase the storage capacity of the lake sufficiently to store therein the ordinary flow of its water-shed.

The storage capacity of the lake was increased by removing the old dam, which was 9 ft. high, and rebuilding another, 17 ft. high, with a spillway 2 ft. higher and foundation 6 ft. lower than the old structure, and by lowering the bed of the outlet to conform to the new grade made necessary by lowering the gates of the dam.

The new dam is 145 ft. long and built of rubble masonry with a face of broken range ashlar, on a platform constructed of sawed hemlock timbers 10 x 12 ins., bedded in clay and concrete and covered with 3-in. planking. A general plan of the dam, with sections through the bulkhead and spillway, is shown in Fig. 1. Extending through the masonry are six rectangular cast-iron sluices, each having a clear opening 3

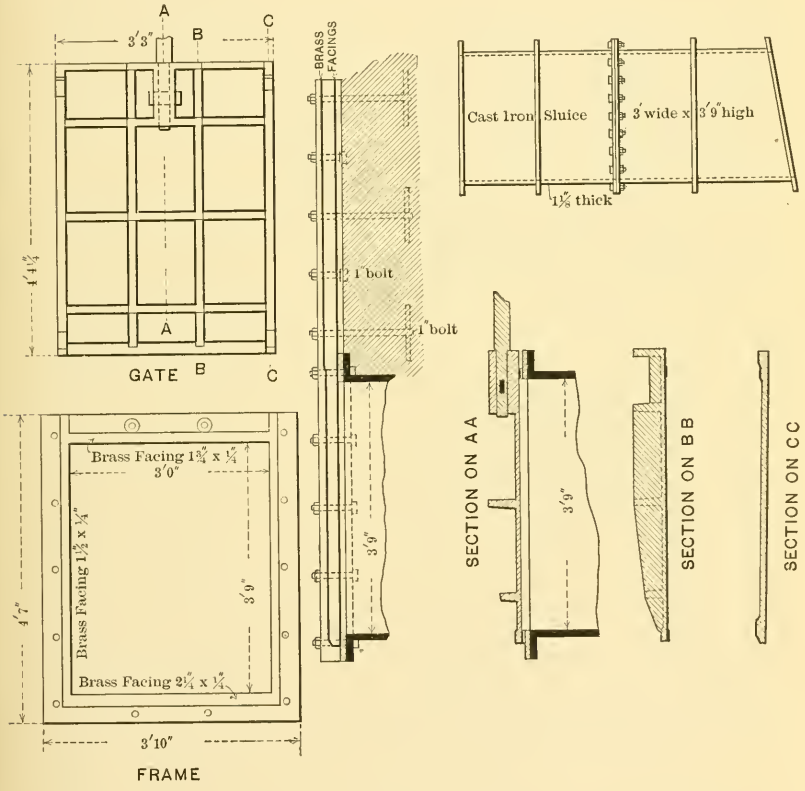


Fig. 2.

ft. in width by 3 ft. 9 ins. in height. The upper end of each is fitted with a brass-faced frame in which slides the cast-iron sluice gate, operated by a hand wheel attached to the 2½-in. screw stem. Details of the sluice and gate are shown in Fig. 2.

A frame gate house, 9 x 32 ft., was built on the dam. The gates control the flow of water in the outlet to supply the canal, and will

permit of the lake being drawn down 17 ft. below its new high water-mark.

A coffer-dam inclosing the westerly end of the old dam was constructed of two rows of machine-driven hard wood piles, 5 x 10 ins., from 16 to 20 ft. long and 6 ft. apart. Waling pieces 8 ins. square were placed on the outside, both at the top and bottom, and connected with $1\frac{1}{8}$-in. tie rods spaced 8 ft. apart. The space between the rows of piles was filled with puddled clay.

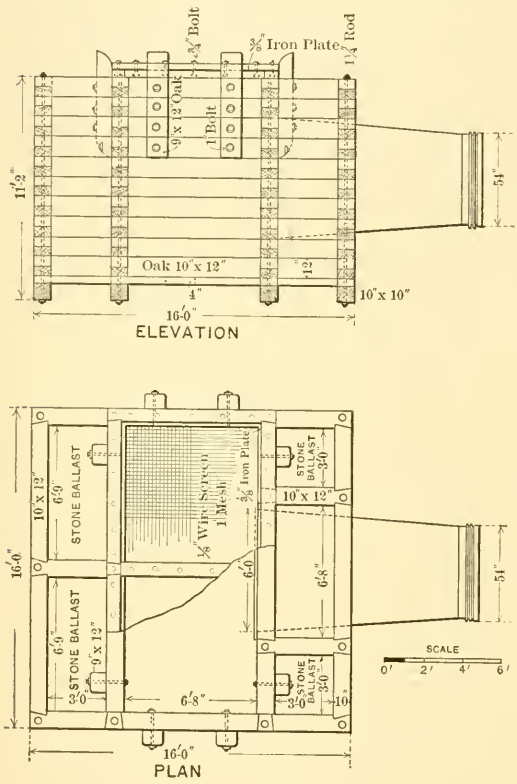


Fig. 3.

The westerly end of the new dam was built within the space inclosed by the coffer-dam. The new dam was located parallel to and on the upper side of the old structure, and the latter was not taken down until the corresponding portion of the new dam was completed. During the progress of the work on this section the water was carried through a gate at the easterly end of the old dam into the outlet to supply the canal. After the first section of the dam was finished, the water was turned through the gates of the new dam, and a coffer-dam was constructed inclosing the easterly end of the old structure and the pit for the foundation of the remainder of the dam.

The new water-works consist of a crib and 6 419 ft. of 54-in. riveted steel intake pipe in Skaneateles Lake; a gate house on its shore; a con-

duit line of 19¾ miles of 30-in. cast-iron pipe; a distributing reservoir near the city with a capacity of 121 000 000 galls., and improvements in the distributing system throughout the city.

After a complete hydrographical survey, including over 3 000 soundings in lines extending southerly from the dam across the lake for 1½ miles, the crib was located in 40 ft. of water, and 6 419 ft. from the gate house on the shore. The crib is 16 ft. square by 12 ft. high, and is shown in Fig. 3. It rests on a foundation of small stone. Its sides are formed of 10 x 12-in. oak timbers, framed and dovetailed together and secured by 1¼-in. tie rods running through the timbers from top to bottom. Partitions, also of oak timbers, divide the crib into six compartments. The water enters the top of the middle chamber through a copper-wire screen of 1-in. mesh, and is conducted through a steel reducer to the intake pipe, the connection being made by a flexible joint. The reducer pipe, surrounded by concrete, occupies one compartment and the other four are filled with stone ballast.

The intake pipe is 54 ins. in diameter and made of ⅜-in. steel plates, weighing 15 lbs. per square foot. Each plate forms a 6-ft. length of pipe, slightly larger at one end, so that it could be telescope-jointed. Five pieces riveted together with 2½-in. lap formed a section 29 ft. 2 ins. long. These sections, after being coated with asphalt, were loaded on platform cars, three lengths to the car, and shipped to the lake.

The coating used was a mixture of the Los Angeles Oil-Burning Supply Company's dry asphalt, grade "D," and their liquid asphalt, grade "G." In heating the mixture, it was necessary to use 3 parts of "D" to 1 part of "G"; once heated, it was necessary to add the grade "G" only. The pipe was completely submerged in the bath, heated to a temperature of 280° Fahr., and left therein about 20 minutes or long enough for the steel to attain the temperature of the mixture. The pipe was then withdrawn, and after the coat was allowed to stiffen, it was again submerged for a few moments to thicken the coat.

On the lake shore, four of the 29-ft. lengths were riveted together, making a section 116 ft. long. Each section was provided with a steel spigot on one end, and a cast-iron hub on the other. The hub was provided with 20 steel hook-bolts 1½ ins. in diameter, with hexagonal nuts. A gasket of 1-in. soft lead pipe, weighing 2 lbs. per foot, was placed around the pipe against the steel spigot, while back of this as

a follower and bearing for the hook-bolts, was a hoop of $\frac{7}{8}$ x 1-in. wrought iron (see Fig. 4).

The flexible joints were made by jointing two short pieces of pipe together, one piece being tapering in diameter and 3 ft. long, the other being a straight piece 4 ft. long, upon the end of which was riveted a machine-faced cast-iron ball or zone. Two 4-in. channel irons were riveted on the inside of the larger end of the tapering piece which fitted over the ball. The channels were run full of hot lead against the ball making the flexible joint, which is capable of a deflection of 12° in any direction from the axis of the pipe (see Fig. 4).

The pipe was laid in a trench in the lake for a distance of about 4 500 ft. The trench was 10 ft. wide at the bottom, and at the shore it was 11 ft. deep, the depth gradually decreasing until the bottom of the trench intersected that of the lake. From this point to the crib the pipe was laid on the bottom of the lake. The excavation was made with a dipper dredge, the material being mostly a hard, red clay. Before the pipe was taken from the skids on the lake shore, the ends of a 116-ft. section were closed by oiled canvas bulkheads, rolled into the water, and floated between the sections of a catamaran which had been placed in position over the pipe trench.

The catamaran was 95 ft. long, 30 ft. wide, and 6 ft. deep. The sections were 12 ft. wide with a 6-ft. space between them for lowering the pipe. The sides and ends were formed of six 5 x 12-in. timbers placed one on top of the other and secured by drift-bolts. Each section was covered with plank. The catamaran was held in place by spud piles at each corner. Ropes from swinging booms were attached to the pipe, the bulkheads removed, and the pipe lowered to the bottom of the lake. Each length of pipe was supported on two timber foundations placed 19 ft. either way from the joint. These timber foundations were secured to the pipe before it was lowered and were held in place by hooks connecting the foundation platform to a saddle piece fitting over the top of the pipe. When the pipe was finally in position, the saddle pieces were removed and used for the next length. Fig. 5 shows this arrangement in detail. A diver guided the spigot into the bell of the last pipe laid, adjusted the hook bolts, and, by screwing up the nuts on the same, upset the lead pipe completing the joint.

In building the gate house the pipe was laid through the wall to the intake well, and the trench and pipe extended back toward the lake for

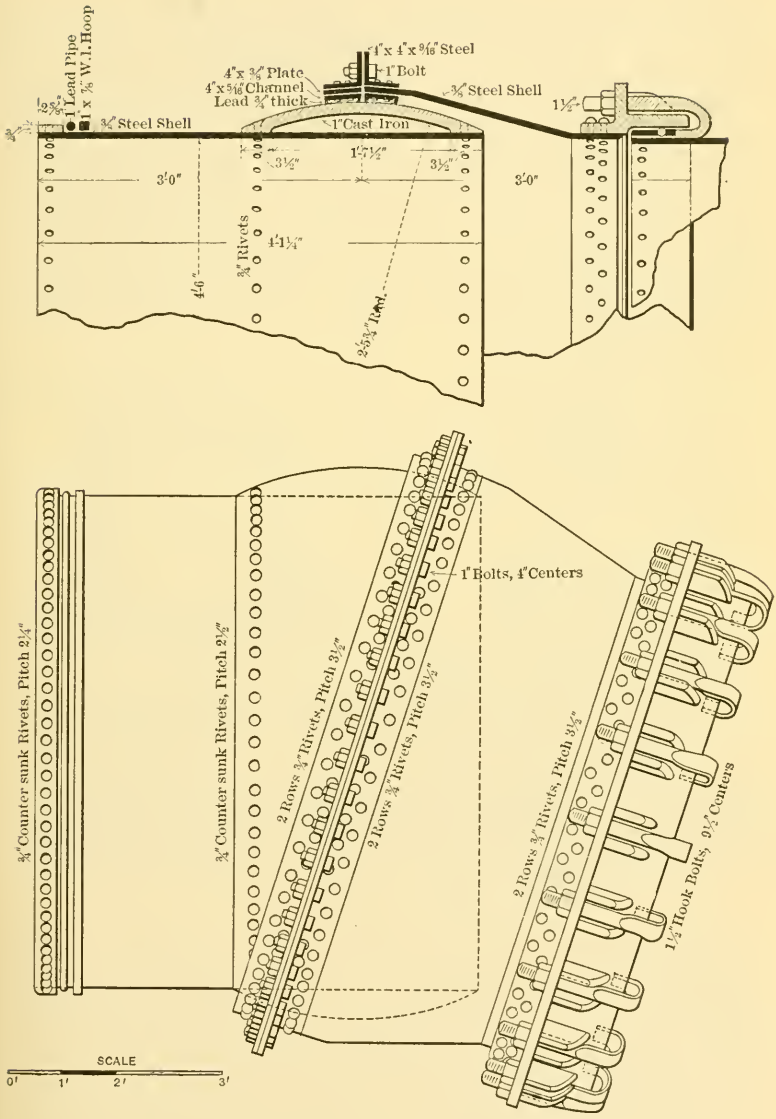

1" Lead Pipe
1 x ⅞ W.I. Hoop
4" x 4" x 9/16" Steel
1" Bolt
4" x ⅜" Plate
4" x 5/16" Channel
Lead ¾" thick
⅜" Steel Shell
¾" Steel Shell
1" Cast Iron
1½"
3'-0"
3½"
1'-7½"
3½"
3'-0"
¾" Rivets
4'-1¼"
4'-6"
2'-5¾" Rad.
1" Bolts, 4" Centers
¾" Counter sunk Rivets, Pitch 2¼"
¾" Counter sunk Rivets, Pitch 2½"
2 Rows ¾" Rivets, Pitch 3½"
2 Rows ¾" Rivets, Pitch 3½"
2 Rows ¾" Rivets, Pitch 3½"
1½" Hook Bolts, 9½" Centers
SCALE
0' 1' 2' 3'

FIG. 4.

a distance of 41 ft. This part of the trench was then filled to prevent the water from reaching the walls of the gate house, and the excavation and pipe laying was continued into the lake to meet the pipe from the crib. The final joint was of two spigot ends joined under water by means of a sleeve joint of special design, shown in detail in Fig. 5. It consisted of a piece of steel pipe 3 ft. in length and of sufficient diameter to pass over the ends of the pipes to be connected, and was provided at each end with hook bolts connected by sleeve nuts. The joint was packed with hemp rope saturated with white lead. When the nuts were screwed up, the hook-bolts drew an iron follower against the packing and made a tight joint.

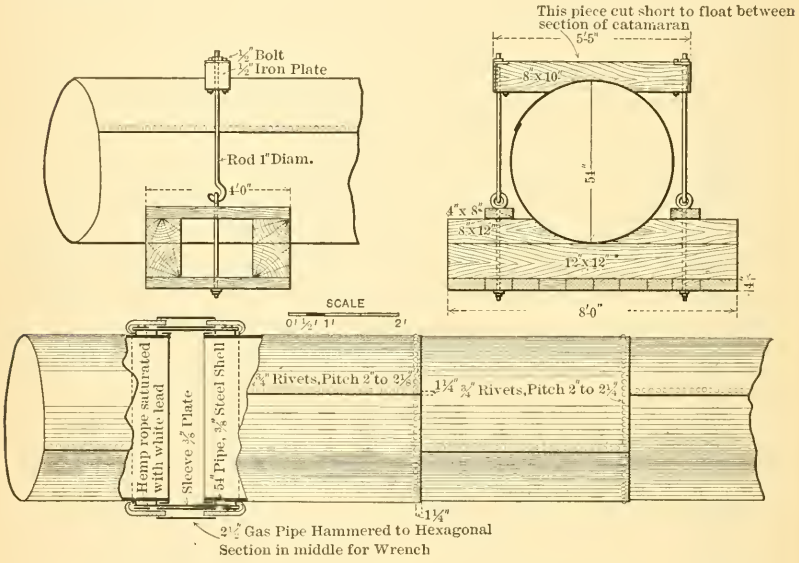


FIG. 5.

The steel pipe was made at Groton, N. Y., by the Groton Bridge and Manufacturing Company. It cost, delivered on the skids at the lake, $8 80 per lineal foot, including the seven flexible joints. The laying (exclusive of the trenching) cost $2 50 per foot.

The gate house contains an intake and screen well each 25 ft. deep. Through the wall separating the wells are two cast-iron sluices each 2 ft. 6 ins. wide by 4 ft. 6 ins. high in the clear, and provided with cast-iron gates similar in construction and operation to those used at the dam. The fish screens are made of $\frac{1}{10}$-in. copper wire, $\frac{1}{4}$-in. mesh, with

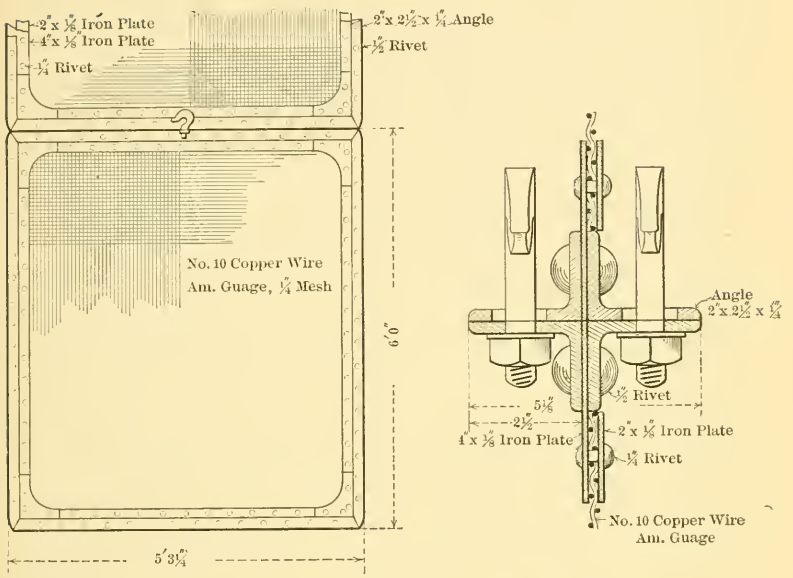


FIG. 6.

wrought iron frames (Fig. 6). A cast-iron reducer pipe, 36 ins. square on one end and reducing to a circle 30 ins. in diameter, connects this well and the 30-in. conduit line. The square end of the reducer is fitted with a sluice gate operated in the same manner as the others.

A 4-in. vent pipe was connected to the first length of 30-in. pipe and carried up and through the wall to the inside of the gate house. The walls are of rubble masonry with vitrified brick facing. The grooves for screens and stop plank were formed of dimension cut stone set in the wall. A longitudinal section and plan of the gate house walls are shown in Fig. 7.

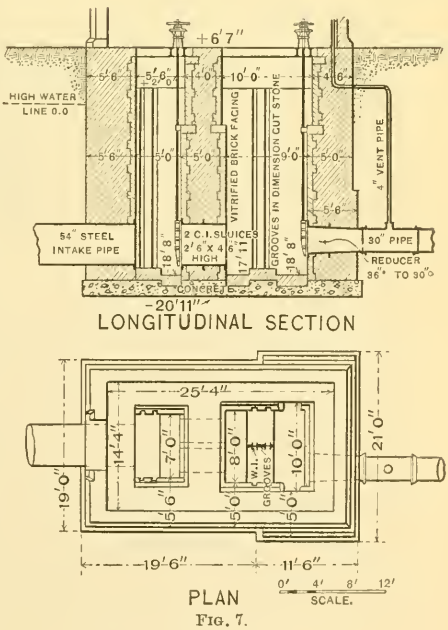


FIG. 7.

The superstructure is of broken range ashlar and pressed brick similar to the gate house at the reservoir.

A means of computing and regulating the amount of water flowing into the conduit is afforded by opening the gate between the wells to a fixed height and adjusting the head of water against that gate by operating the gate in the screen well.

Extensive surveys were made of all the practicable routes for the conduit line from the lake to the site of the reservoir near the city. The topographical notes were obtained by using a slope board, which was a home-made instrument consisting of a smooth board about 10 x 20 ins. of convenient shape to be easily handled, and having on its face an

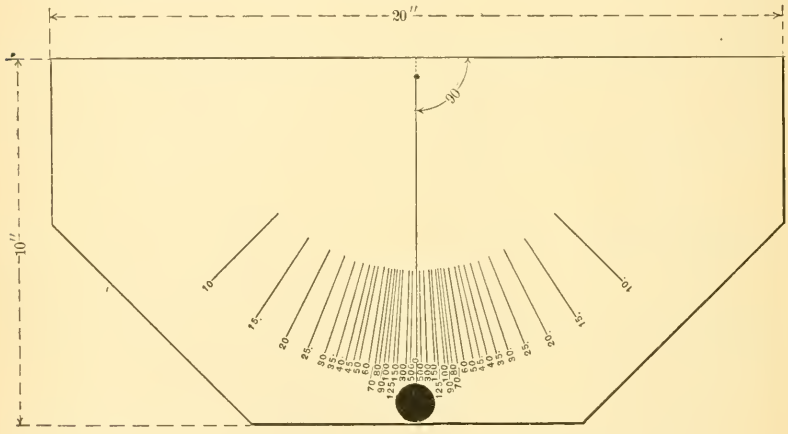


FIG. 8.

arc graduated as explained below. From the subscribing center of this arc a fine line was suspended, to which was attached a circular flat weight that would swing just below the graduations. When the top edge of the board was held level the pendulum line would intersect the arc at zero or the initial point of graduation (see Fig. 8).

In using this instrument the topographer sights along the top of the board and brings it parallel to the slope of the ground. The line of the pendulum will then intersect the graduated arc, and this reading is recorded in his note book. However, instead of indicating the degree of the slope the graduation marks the horizontal distance between contours of 10 ft.

Angles on each side of the initial point of—

1° 09′	are marked	500	11° 19′	are marked	50
1° 54′	"	300	12° 33′	"	45
3° 49′	"	150	14° 03′	"	40
4° 35′	"	125	15° 57′	"	35
5° 43′	"	100	18° 26′	"	30
6° 21′	"	90	21° 49′	"	25
7° 08′	"	80	26° 36′	"	20
8° 08′	"	70	33° 40′	"	15
9° 27′	"	60	45° 00′	"	10

With this graduation, when the top of the board is parallel with the surface of the ground, the line of the pendulum will indicate the horizontal distance between 10-ft. contours of that slope. Thus an angle of 14° 03′ will read 40 ft., and if that slope extends 200 ft. the proper entry in the note book will be $\frac{10 \text{ in } 40}{200}$ prefixed by a plus or minus sign, according as the slope was ascending or descending from the line.

This graduation of the arc greatly facilitates the work of plotting the notes. Knowing the elevation of any station, the location of the first contour from the line is obtained by multiplying the reading of the board by $\frac{1}{10}$ the vertical distance to the first contour. After the first contour is located or plotted, the others are measured off direct until the entire length of slope is consumed. From the data thus obtained a contour map was made which gave in detail the features of the country for several hundred feet each side of the lines surveyed. On this map the pipe line was projected, and from the notes of the projection the final location was staked out on the ground. Great care was exercised in making this projection, and the number of summits and depressions in the grade was reduced to a minimum. This was accomplished without any material increase in the length of the line.

All horizontal deflections in the conduit line were made by using straight pipe laid to regular circular curves, the maximum being 20° curves on the first part of the line where the pressure is light, and 10° curves on the lower parts of the line. The vertical curves were made in the same way, except at the very steep slopes of the Marcellus and the Camillus Ravines, and at Geddes Brook.

The cast-iron pipe was made in lengths to lay 12 ft. with hub and spigot, as shown by Fig. 9. All straight pipes were cast vertically in dry sand molds or flasks, hub end down. All pipes and castings were thoroughly inspected by men who acted under direct orders from the author. Each pipe, after being cleaned and examined, was dipped in tar in the usual manner, and subjected to a hydraulic pressure of 300 lbs. per square inch, and while under pressure was required to stand smart blows with a hammer over its exterior surface. Test bars were cast from each heat and required to have a tensile strength of at least 16 000 lbs. to the square inch.

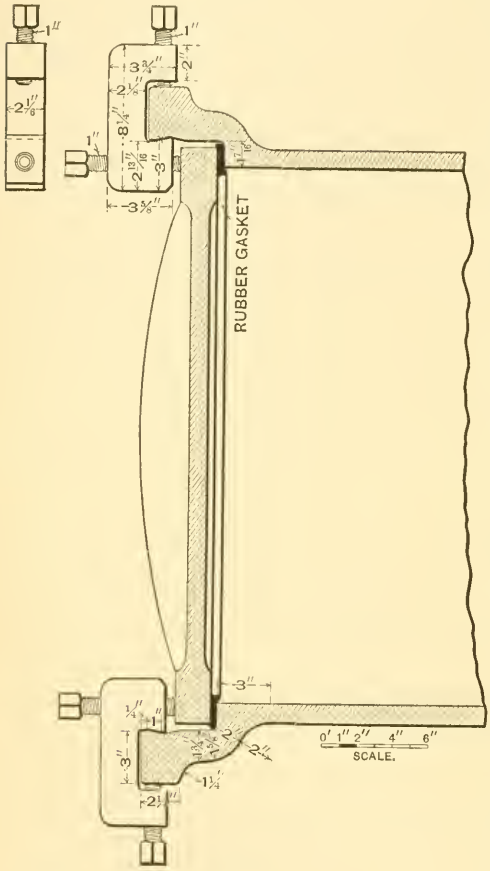


FIG. 9.

The following table shows the thickness of the 30-in. cast-iron pipe used under the different heads.

Class.	Thickness.	Maximum head.
	Inches.	Feet.
A	$\frac{7}{8}$	165
B	$\frac{15}{16}$	205
C	1	255
D	$1\frac{1}{16}$	295
E	$1\frac{1}{8}$	335
F	$1\frac{3}{16}$	375
G	$1\frac{1}{4}$	415
H	$1\frac{5}{16}$	460
I	$1\frac{3}{8}$	475

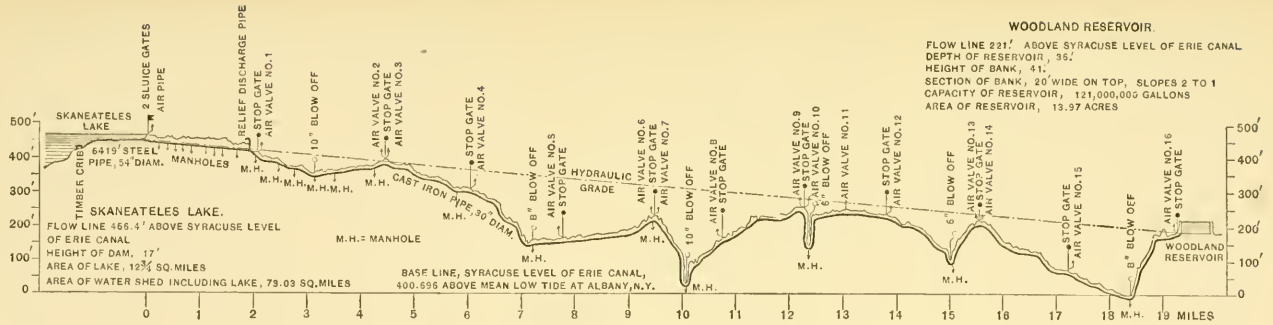

WOODLAND RESERVOIR.
FLOW LINE 221.' ABOVE SYRACUSE LEVEL OF ERIE CANAL.
DEPTH OF RESERVOIR, 35.'
HEIGHT OF BANK, 41.'
SECTION OF BANK, 20' WIDE ON TOP, SLOPES 2 TO 1
CAPACITY OF RESERVOIR, 121,000,000 GALLONS
AREA OF RESERVOIR, 13.97 ACRES
SKANEATELES LAKE
2 SLUICE GATES
AIR PIPE
RELIEF DISCHARGE PIPE
STOP GATE
AIR VALVE NO. 1
10" BLOW OFF
AIR VALVE NO. 2
STOP GATE
AIR VALVE NO. 3
STOP GATE
AIR VALVE NO. 4
8" BLOW OFF
AIR VALVE NO. 5
STOP GATE
HYDRAULIC GRADE
AIR VALVE NO. 6
STOP GATE
AIR VALVE NO. 7
10" BLOW OFF
AIR VALVE NO. 8
STOP GATE
AIR VALVE NO. 9
STOP GATE
AIR VALVE NO. 10
6" BLOW OFF
AIR VALVE NO. 11
STOP GATE
AIR VALVE NO. 12
6" BLOW OFF
AIR VALVE NO. 13
STOP GATE
AIR VALVE NO. 14
STOP GATE
AIR VALVE NO. 15
8" BLOW OFF
AIR VALVE NO. 16
STOP GATE
WOODLAND RESERVOIR
TIMBER CRIB
6419' STEEL PIPE, 54" DIAM.
MANHOLES
M.H.
CAST IRON PIPE, 30" DIAM.
SKANEATELES LAKE.
FLOW LINE 466.4' ABOVE SYRACUSE LEVEL OF ERIE CANAL
HEIGHT OF DAM, 17'
AREA OF LAKE, 12¾ SQ. MILES
AREA OF WATER SHED INCLUDING LAKE, 73.03 SQ. MILES
M.H.= MANHOLE
BASE LINE, SYRACUSE LEVEL OF ERIE CANAL,
400.696 ABOVE MEAN LOW TIDE AT ALBANY, N.Y.
500'
400'
300'
200'
100'
0
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 M.H. 19 MILES

FIG. 10.

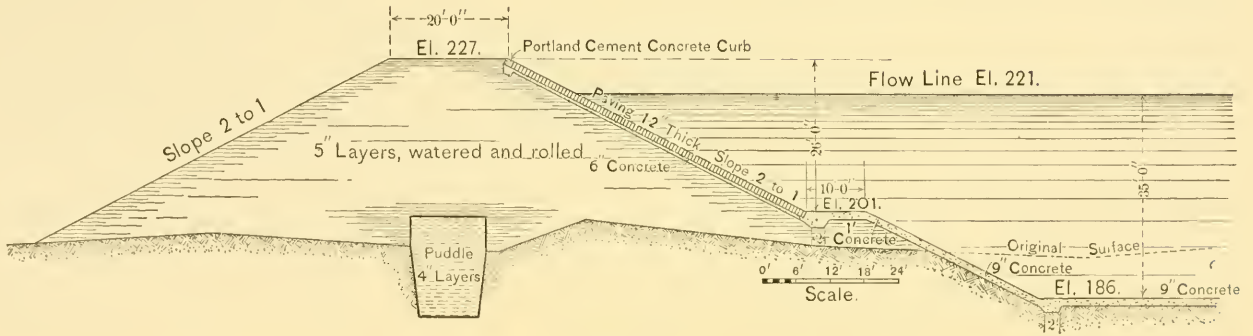

20'-0"
El. 227.
Portland Cement Concrete Curb
Flow Line El. 221.
Slope 2 to 1
5" Layers, watered and rolled
6" Concrete
Paving 12" Thick
Slope 2 to 1
25'-0"
10'-0"
El. 201.
1' Concrete
Original Surface
9" Concrete
El. 186.
9" Concrete
35'-0"
Puddle
4 Layers
0' 6' 12' 18' 24'
Scale.

FIG. 11.

On the conduit line there was used 18 367 tons of straight pipe, 135 tons of curved pipe, and 156 tons of special castings, furnished by Dennis Long & Company, of Louisville, Ky.; the Radford Pipe and Foundry Company, of Anniston, Ala.; the McNeal Pipe and Foundry Company, of Burlington, N. J., and R. D. Wood & Company, of Philadelphia, Pa.

The conduit line was divided into four sections, numbered 1, 2, 3 and 4 respectively, beginning at the lake, each section being about 5 miles long. All the work on each section was let separately by contract, and each was completed satisfactorily, with the exception of Section No. 1. The contractor abandoned this, the most difficult on the line, and the work was completed by the Water Board.

Previous to the letting of the contracts, test pits were dug along the line at intervals of 590 or 600 ft. These were carried down to grade unless rock was reached, and were left open for the inspection

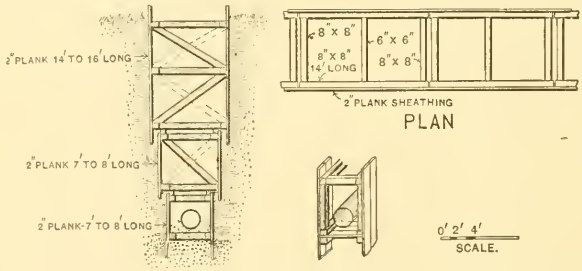


FIG. 12.

of the bidders. The trenching and refilling were included in one price per cubic yard, and the pipe laying, joining, and calking, including the furnishing of lead and yarn, were paid for by the lineal foot. The minimum depth of the lead joint was 2½ ins. The minimum width of the right of way secured was 60 ft.

In general the trench was 4½ ft. wide on the bottom, with side slopes of 1 horizontal to 6 vertical. The minimum covering over the pipe was 4½ ft. All pipes were laid to line and grade from stakes set in the bottom of the trench and were supported on blocks and wedges.

A glance at the profile (Fig. 10, page 37) will show that in order to keep the conduit below the hydraulic grade line, deep cutting was necessary on the first 2 miles. The line there follows the west bank of the Skaneateles outlet and then crosses under that stream. The average depth of the trench on 1 mile was 20 ft., with a maximum depth of 30

feet. Much difficulty was experienced with quicksand and water. Fig. 12 shows the style of sheeting used on this part of the work. It was necessary to put in a flooring braced against and held down by the sheeting in order to keep the quicksand from rising and filling the trench. In many cases the ground back of the sheeting sunk to a depth of 12 ft., carrying the sheeting with it. This required constant care and work adjusting and putting in additional cross-pieces to keep the trench open. The timber used was spruce, 8 x 8 ins., with 2-in. planking of the same material. The pipe was laid on blocks and wedges placed on the flooring.

At the lower end of the deep cutting on Section No. 1 a 20-in. relief pipe was put in as a precaution against a water ram on the pipe through the quicksand. Should an accident occur at this place it might take a month's time to repair the break. The relief pipe was carried up the side of the valley to a height of 31 ft. above the hydraulic grade line and to an elevation of 456 ft. The 20-in. pipe at this elevation terminates with two 90° bends where it will discharge its water into a 3-ft. length of 30-in. pipe set in an upright position. From the bottom of this 30-in. pipe a line of 12-in. pipe is laid to the creek. A screen is placed over the upper end of the 12-in. pipe to prevent frogs, etc., from entering the main conduit line. The upper end of the relief pipe is inclosed in a plain brick house.

Prominent and principal features of the conduit line were the crossings of the Marcellus and the Camillus Ravines. Marcellus Ravine is 80 ft. deep, with sides sloping at angles of 40° and 48° respectively. The bottom is flat and 500 ft. wide. Nine Mile Creek, the outlet of Otisco Lake, flows through this valley, and at the crossing of the conduit line it is in two channels, the larger being about 25 ft. wide and 3 ft. deep, and located at the foot of the eastern slope of the ravine. The water was diverted from the westerly into the easterly channel until the pipe was laid from the west up to the bank of the eastern channel. All the water was then diverted into the westerly channel, and the pipe laying continued under and across the easterly channel and up the eastern slope at an angle of 42°. The vertical curves were made by using special curved pipe 8 ft. long, with a radius of 34½ ft. These curves were used at the top and bottom of the slopes at each side of the ravine.

Camillus Ravine is 100 ft. deep and only 20 ft. wide at the bottom.

The sides are very steep, the westerly slope being at an angle of 45°, upon which the pipe is laid at an angle of 42°. In this ravine is a small stream of water, which, however, disappears some distance above the place where the conduit line crosses. In excavating the trench across the bottom, shale rock was found to a depth of 6 ft. Below this was a bed of quicksand and water 12 ft. deep. A row of piles was driven, capped with timber and concreted, which construction formed a firm foundation for the pipe. The concrete was carried up, over and around the pipe and covered with flag stones, to prevent a washout in time of high water in the stream. The vertical curves at the top and bottom were made by the use of special curved pipe, the same as at Marcellus Ravine.

One of the various methods for setting the pipes to line and grade consisted in the use of a board template arranged to fit across the inside end of the pipe, so that the upper edge of the template would coincide with the horizontal diameter. This template was placed in the end of the last pipe laid. The center of the top edge of the board would then mark the center of the pipe for line and grade. Some distance ahead in the trench, a target was set in line and to grade for the center of the pipe. The pipe inspector, inside the last pipe laid, directed the adjustment of the next pipe until the center of its template was brought in line with that of the fixed one and the target. The pipe was then held in position by the blocks and wedges. With this method, under the most favorable conditions, 102 lengths of 30-in. pipe were placed in position in the trench in one day.

Eleven 30-in. gates were used on the conduit line, each with two 8-in. by-passes. These gates were made by the Eddy Valve Company, of Waterford, N. Y., from a special design for this work.

Sixteen 4-in. air valves were placed on the line as indicated on the profile. These are also of a special design, and have proved very satisfactory thus far. They are constructed to work automatically, and will open at 50 lbs. pressure to let air into the pipe while being emptied.

Their gate and seat are similar to like parts of the Matthew's hydrant, and are held together by a ¼-in. brass wire spring. The gate can also be opened at any time by a screw stem, and as a precaution against accident to the spring which draws the gate back to its seat, an extra inside screw is provided to be screwed into the gate, and thus

raise it (Fig. 13). An ordinary 4-in. flanged valve is placed between the air valve and the conduit, to be used in case it is necessary to make any repairs to the air valve. A plain brick building was built over each of the 30-in. gates and air valves on the conduit line.

Manholes were placed on the conduit line at the low points and near the valves, while through the deep cutting on Section No. 1 they were placed more frequently. The manhole proper is a special pipe casting 6 ft. long, having an elliptical opening 12 x 16 ins. in the clear. The opening is closed by a cast-iron plate on the inside held in position against a flange by bolts through two saddle-pieces resting on the outside of the casting. The opposing surfaces of the plate and special were ordinary cast faced. Two layers of flat lamp wick, 1 in. wide, saturated with white lead, were used for gasket, and made a perfectly tight joint.

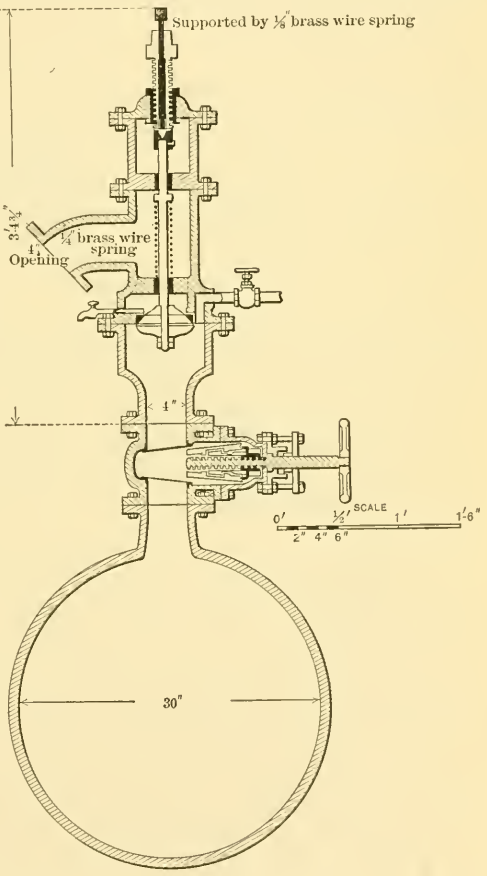


Fig. 13.

In deep cuttings circular brick vaults were built around the manholes and carried up to within 3 ft. of the surface, where they were closed with a cast-iron cap, which was carefully referenced before being covered with the earth.

A blow-off branch was laid at each of the six depressions in the conduit line. A special casting 6 ft. long, with hubs at both ends and

a small branch opening on the side, was laid at the lowest places in the grade. From this lateral opening pipes were laid to a discharge chamber.

Between the chamber and the special, a gate was inserted in the line of the small pipe. The gate was inclosed by a circular brick vault, capped with a 10-in. course of cut stone. There was an opening 3 ft. square in the center of the cap stone, which was covered with iron doors, with hinges bolted to the stone (see Fig. 14).

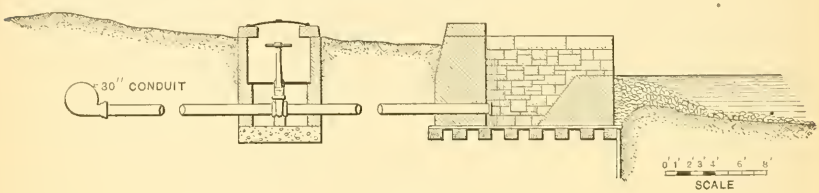


FIG. 14.

The discharge chamber was built of ashlar faced stone masonry resting on a timber platform foundation. The interior of the chamber was 4 x 7 x 4 ft. deep from the top of the spillway.

At the easterly end of Section No. 2 the conduit is laid along a steep wooded slope commonly known as the "Alps." At many places it is at an angle of 45° and rises to an elevation of about 500 ft. above its base.

About midway up the slope on the upper side of the conduit line a roadway 6 000 ft. in length had to be built before the pipe or other material could be delivered. For about 2 000 ft. the cutting for the roadway was through solid rock which cropped out at the surface. In excavating for the pipe trench, solid rock was encountered for 1 800 ft. Owing to the irregular character of the slope and the determination to keep the conduit to a nearly direct line and proper grade, the cutting at some points was carried to a depth of 19 ft., while at others it was very light. At the latter-mentioned places retaining walls, six in number, were built to support the embankment over the pipe.

The walls had a standard width of 3 ft. on top with a batter of 3 ins. to the foot on the face. They were laid dry and were of good quality gray limestone, roughly hammer dressed. In the six walls there were 2 500 cu. yds. of masonry.

As a precautionary measure, after the pipe was laid, orchard grass

seed was sown over the pipe trench and on the embankments, to prevent the washing away of the newly placed earth.

After the pipe was laid and calked, a hydraulic pressure test was applied for the twofold purpose of revealing imperfect pipe and faulty joints. If no defects were found, the contractor was then allowed to refill the trench on that part of the work. Pumping stations for testing the pipe were established on each of the four sections in the immediate vicinity of blow-offs, as they were convenient to water and the blow-off special furnished an opening through which water could be pumped into the conduit.

The open ends of the conduit were closed with false heads made especially for this work. They were cast-iron plates, machine-faced on the inner side, and fitted snugly into the hub end of the pipe. A heavy rubber gasket was used between the plate and inner face of the hub. Each head was secured with 12 clamps (see Fig. 9). Heavy hard wood timbers and jack screws formed a brace against the end of the pipe and prevented it from being shoved ahead in the trench by the pressure. In each of the false heads two openings of 1 in. and 2 ins., respectively, were cut. Into the larger one a short piece of wrought-iron pipe, with a valve attached, was screwed. This provided for the escape of air. The smaller aperture was for the pressure gauge.

When the pipe was filled and the required pressure attained, two inspectors, one on each side of the pipe, examined the joints for sweats and leaks, and likewise the body of the pipe itself for cracks or spongy iron. If sweats or leaks were found, the pressure was relieved by opening a valve and letting a small quantity of water out. The joints were then recalked and the test pressure again put on. Rarely a third application of pressure took place. Occasionally pin holes were found in the metal, but these, with the exception of a few cases, "took up" themselves by oxidization, which filled the minute holes. Two, of which there was some doubt of the rust closing, were tapped and plugs inserted. Some three or four defective pipes were discovered and taken out. After the completion of the test the water in the pipe was drawn down so that its surface was on a level with the bottom of the last pipe and the balance of the water was left in the pipe to be used for the next test.

Grades were ascending in either direction from the blow-off special, and all that was necessary when a subsequent test took place was to fill the newly laid pipe.

The pumping station in Marcellus Ravine was at the westerly end of Section No. 3. From here tests were made to the connection with Section No. 4, a distance of 4 miles. Two pumps were used here, one with a large cylinder for filling the pipe, which it would do very rapidly, and a second one with a very small cylinder, capable of developing 500 lbs. per square inch for furnishing pressure for the test. It might be well to mention the fact that the larger pump was tried for the entire work of testing, but it proved inefficient. It would work easily against the head necessary to raise the water in the pipe, but above that the pressure shown by the gauge fluctuated so that the pump was abandoned for that part of the work and the smaller one used entirely. This arrangement gave most satisfactory results.

The minimum pressure allowed on any part of the work was 100 lbs. per square inch. But in every instance each pipe was subjected to a pressure at least 40 lbs. in excess of the stated pressure. The contractors strongly urged that the tests be omitted. They spoke of their calkers in the highest terms, declaring that none of their joints would leak. The first test developed many. After that each calker marked the joints he made, and each took a pride in his work. The best testimony of the necessity of tests is the fact that no leaks have been discovered in the line since the water was turned on, nine months ago.

On Friday, June 29th, 1894, at 10.11 A. M., the gates in the well house at Skaneateles were opened, and water for the first time began to flow from Skaneateles Lake to Syracuse. The draft from the lake to fill the conduit was regulated to 1 000 000 galls. in 24 hours.

While the turning on of the water was the practical consummation for which the city had been contending for years, it took place without any ceremony. In fact it was done as quietly as possible as fears were entertained that some legal obstacle might be interposed at the last moment.

In filling the conduit line each gate house and blow-off was visited in advance of the water, and its arrival and passage waited for. The approach of water was heralded in advance by the escape of the compressed air through the air valves. It would begin with the least perceptible murmur and continue to increase in loudness until it assumed a roar like that made by steam escaping from the exhaust valve of a steam boiler.

The gate at each of the blow-offs was opened in turn and water al-

lowed to waste until the last vestige of sediment from the conduit line had disappeared. As soon as the intermediate sections between the gate houses were filled, men were put on patrolling the conduit to discover breaks or leaks should any occur. At 4.12 in the afternoon of July 3d water reached the north stop gate house of the new reservoir and a few minutes later rushed with a roar into the old distributing reservoir.

The work of filling the pipe occupied nearly 4½ days and was carried on without cessation. An hour's sleep in some fence corner or on top of some blow-off chamber had to suffice for rest. Meals were eaten at irregular intervals; sometimes breakfast was eaten at noon and oftener not at all.

This part of the work had its amusing as well as serious side. One of the watchmen, stationed at night in the "Alps," became scared at an owl's hooting, climbed a tree and staid there until the return of a patrolman. Another man, who had been left in charge of a gate house without instructions as to meals, staid there three days and subsisted on a few crackers and a piece of cheese.

The new distributing reservoir which is now being constructed is located just outside the city limits about 2 miles southwesterly from the center of the city. The site is on a summit in a valley with long and rounding hills on either side, that on the west having an elevation of 280 ft. and the one of the east 220 ft. The reservoir is formed by building earth embankments across the valley at the north and south ends and along and over the lower hill on the east.

Preliminary to the selection of the site and the location of the embankments, the underlying earth was examined. This was accomplished by driving a 2-in. wrought iron pipe and drawing it from different depths, bringing up samples of the material. Later on, test pits were dug at different places over the site, some to a depth of 20 ft.

The site was cleared of trees, stumps and roots. The loose top soil was removed and placed in a spoil bank for top dressing the embankment. A pit 12 ft. wide was dug on the line of the center of the embankment to a general depth of 8 ft. This was filled with clay placed in 4-in. courses and puddled in position. The embankment was started on a firm foundation which was thoroughly dampened. It was made of material excavated from the sides and bottom of the

reservoir, which was found to be a mixture of heavy clay with a small amount of gravel. All hard lumps were broken and all stones more than 4 inches in diameter were picked out. The material was leveled off in 5-in. layers by a road scraper, then dampened and rolled by a grooved roller weighing 1 200 lbs. per lineal foot.

The embankment is 20 ft. wide on top, with sides sloping 2 to 1.

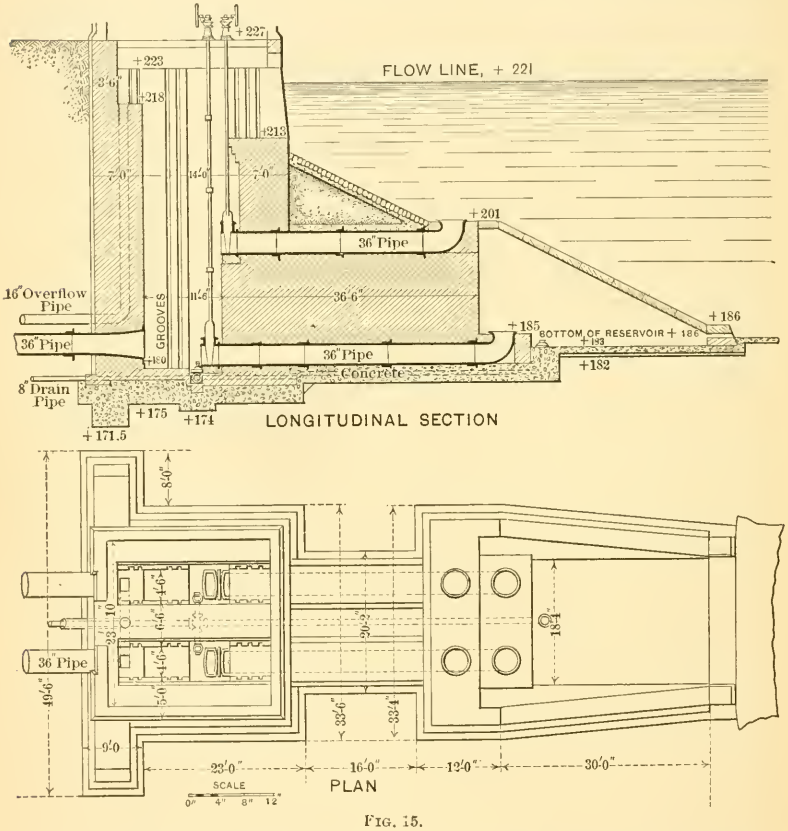


Fig. 15.

On the inner slope there is an offset 10 ft. wide (see Fig. 11, Page 37). The elevations of the various portions of the embankment are shown in Figs. 11 and 15.

The area of the reservoir is 13.97 acres. Its capacity is 121 000 000 galls. with the water 35 ft. deep. The bottom and sides are covered with concrete placed 9 ins. thick over the bottom and lower slopes, 12

ins. thick over the offset, and 6 ins. thick under the paving on the upper slope.

The concrete was composed of one measure of American hydraulic cement, two measures of sand, and three of stone. All the stone was broken by a crusher and was required to be not larger than would pass through a 1½-in. ring in any direction.

The cement was delivered on the work a sufficient time in advance of its use to enable thorough tests to be made. It was required to be of such fineness that 90% would pass through a screen of 2 500 meshes per square inch. When mixed in the ratio of 1 lb. of sand to 1 lb. of

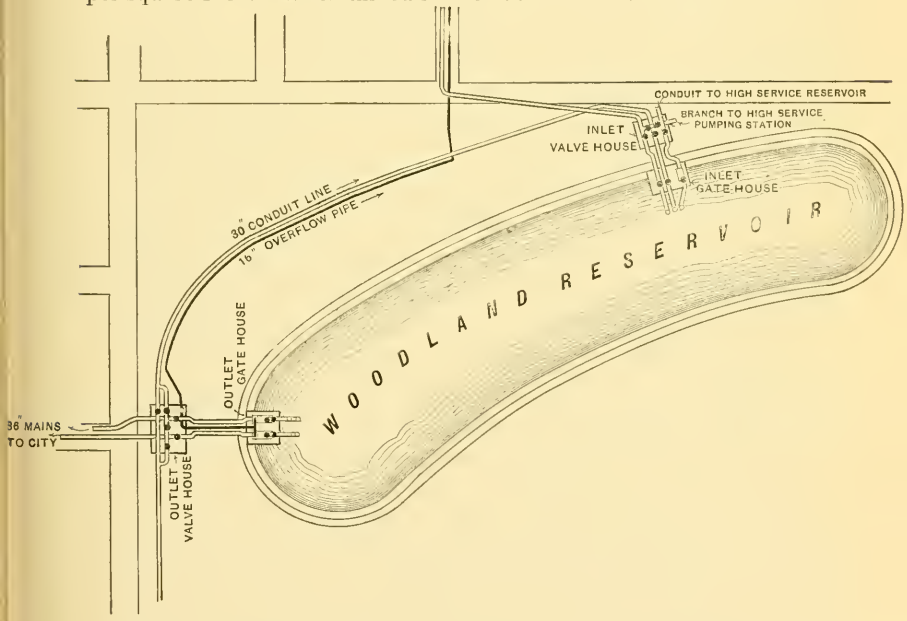


Fig. 16.

cement and exposed one day in air and six days in water, it was required to withstand a tensile strain of not less than 60 lbs. per square inch. Almost the entire quantity of cement used to date was furnished by the F. O. Norton Company and by the Lawrence Cement Company. The average strength of 4 623 briquettes was 100½ lbs. per square inch.

The wall covering the upper slope is 12 ins. thick and is composed of hammer dressed quarry stone laid in full beds of cement mortar on the 6 ins. of concrete. A Portland cement concrete curb 2½ ft. deep and 10 ins. thick is to be laid on top of the slope wall around the reservoir.

In Fig. 15 is shown the general plan of the gate house wells. The walls are of broken range stone with face scabbled to ½-in. projection.

A general plan of the pipes entering and leaving the reservoir, together with the location of the gates and an overflow pipe leading from the wells of the outlet gate house to the old reservoir, is shown in Fig. 16. The 30-in. conduit from the lake passes through the outlet valve house where it is connected with two 36-in. supply mains leading from the reservoir to the city. Valves were put in these connections. The conduit line continues around to the inlet valve house and enters the reservoir through the inlet gate house.

Two other 30-in. mains are also laid through the inlet valve and gate houses. One of these mains is connected with the old pumping station on Onondaga Creek and can be used in case of an accident to the conduit from the lake. The other is to be a supply main to convey water from the reservoir to a high service pump house to be built adjoining the inlet valve house at some future time.

Within the city limits there is a small unoccupied area that is higher than the flow line of the reservoir. Should it ever be desirable to supply this tract with water a high service reservoir might be constructed with a main from the inlet valve house. By operating gates, this reservoir could be supplied by gravity directly from Skaneateles Lake, or by pumping the water from the distributing reservoir, with a high service reservoir at an elevation of 320 ft. and two distributing systems, the entire city would have ample pressure for fire protection.

At the close of the season of 1894, the embankment of the reservoir was completed, and the slope wall was built to a height of 6 ft. above the offset. The reservoir was then filled with water to a depth of 2 ft. above the top of the wall to protect it from the action of the frost. During the winter the city has taken its supply directly from Skaneateles Lake through a connection at the outlet valve house, leaving the surplus water from the lake to flow into the reservoir.

The height of the water in the reservoir, which regulated the pressure on the city mains, was adjusted by operating the gate in the 30-in. main leading from the reservoir to the pumping station on Onondaga Creek. On this line of pipe there is a branch to the old reservoir where the surplus water was discharged.

The reservoir will be emptied of water before the work of completing it is resumed in the spring. The pressure on the city mains will

then be regulated by using the wells of the outlet gate house as a stand pipe. The 16-in. overflow pipe from the wells is of a capacity equal to the 30-in. main from the lake.

The plant of the Syracuse Water Company, as described above, was acquired by the city through condemnation proceedings. The price paid, as awarded by the appraisers, was $850 000.

The Water Board took possession of the works on January 1st, 1892. At that time the pressure on the mains in the lower part of the city was 46 lbs. per square inch. Before increasing this pressure about 14 miles of sheet iron cement-lined pipe and old cast-iron pipe of insufficient size were taken out and replaced by new cast-iron pipe of proper size. In addition there have since been laid 55 miles of new mains, so that at the present time there are 80 miles of street mains and 1 408 hydrants.

The following table gives the standard weight of pipe that will lay 12 ft., and the thickness of the different sizes used in the city distribution, also the thickness of the lead joint.

Diameter of pipe. Inches.	Thickness. Inches.	Weight. Pounds.	Thickness of lead joint. Inches.
4	$\frac{13}{32}$	240	$\frac{5}{16}$ to $\frac{3}{8}$
6	$\frac{7}{16}$	360	"
8	$\frac{7}{16}$	492	"
10	$\frac{1}{2}$	672	"
12	$\frac{9}{16}$	900	"
16	$\frac{21}{32}$	1 356	$\frac{3}{8}$ to $\frac{7}{16}$
20	$\frac{3}{4}$	1 980	"
24	$\frac{27}{32}$	2 664	"
30	$\frac{7}{8}$	3 432	"
30	1	3 948	"
36	$1\frac{1}{16}$	5 004	$\frac{7}{16}$ to $\frac{1}{2}$

In general the pipe has been laid in a line 10 ft. off the center of the street. This leaves the center for a sewer, and the opposite side for a gas pipe.

The water pipe is laid to a grade generally conforming to that of the street. The minimum covering allowed on top of the pipe is 4½ ft. Before refilling the trench the pipe and joints were tested by water pressure, the water being let in from the adjoining mains.

In refilling the trench the earth was thoroughly compacted under and around the pipe to a height of 6 ins. above the top of the pipe. The filling was then wetted down with water from a hose attached to

an adjacent hydrant and the refilling was afterward continued without tamping to within 6 ins. of the surface. The trench was then

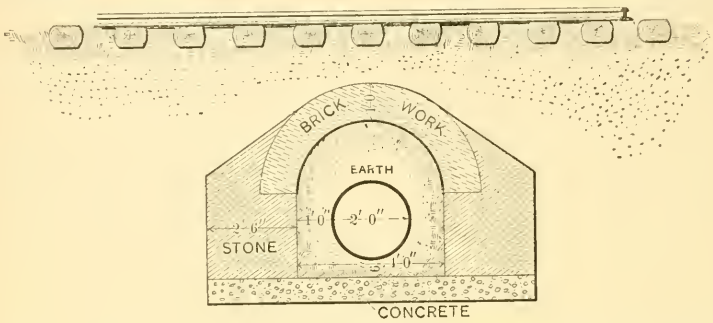


FIG. 17.

flooded with water and allowed to settle. The remainder was put in dry and thoroughly rammed.

This manner of filling the trenches gave good satisfaction, in fact better than where the filling was put in dry and compacted by ramming only.

Where pipes were laid under the tracks of steam railroads, they were arched over with masonry (see Fig. 17). This protects the pipe from the jar of the trains, and the road-bed from damage by water in case of a leak or broken pipe.

All work was done by contract. The trenching and refilling were paid for at one price per linear foot for each of the different sizes of pipe. Placing and calking the pipe, together with specials, gates and hydrants, including the furnishing of lead and

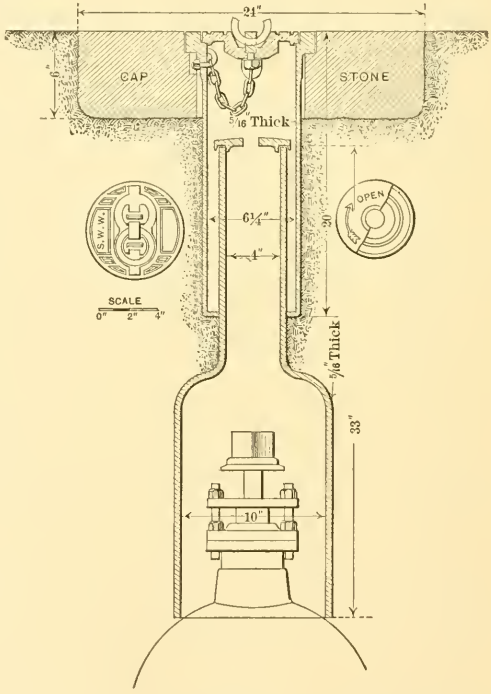


FIG. 18.

yarn, were paid for at one price per lineal foot for each of the different sizes.

The valves were placed in the cross-walks as a rule, the object being to aid in locating them at times when they are wanted in a hurry or when they are covered with snow.

The stems of 12-in. or smaller valves are without gearing and are enclosed by cast-iron boxes (see Fig. 18). The larger valves are geared and are surrounded by brick vaults covered by cast-iron caps (see Fig. 19).

The valve stems are of solid bronze of the following sizes:

36-in.	valve	3 ins. in diameter.
30-in.	"	$2\frac{5}{8}$ ins. "
24-in.	"	$2\frac{3}{8}$ ins. "
20-in.	"	$2\frac{1}{8}$ ins. "
16-in.	"	$1\frac{3}{4}$ ins. "
12-in.	"	$1\frac{5}{8}$ ins. "
10-in.	"	$1\frac{1}{2}$ ins. "
8-in.	"	$1\frac{3}{8}$ ins. "
6-in.	"	$1\frac{1}{4}$ ins. "
4-in.	"	$1\frac{1}{8}$ ins. "

All rubbing surfaces of valves are of non-corrosive metal, brass or bronze.

Hydrants were generally placed not more than 300 ft. apart, and those located at street corners were connected with the larger main. Valves were placed in the hydrant branches where they led from mains 12 ins. or larger in diameter. All hydrant branches were 6 ins. in diameter.

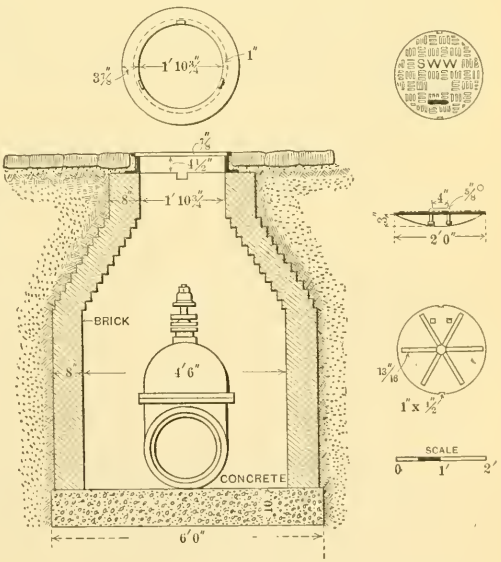


Fig. 19.

Hydrants were required to have an effective discharging area between the branch connection and the hydrant gate not less than that of a 6-in. pipe. The

diameter of the hydrant barrel above the gate was required to be at least $5\frac{1}{2}$ ins., and the valve opening not less than that of a pipe $4\frac{1}{2}$ ins. in diameter.

All hydrants were provided with a brass drip spigot of special design, to which was connected by a wiped joint a 1-in. lead pipe weighing $3\frac{1}{2}$ lbs. per foot. This drains the hydrant barrel to the nearest sewer or drain.

A block of masonry was built back of the base of the hydrants to protect the joints in the branch from the effect of a water ram.

The length of the hydrant barrel from the pavement to the top of the branch pipe is $5\frac{1}{2}$ ft.

During the very severe cold winter just passed none of the hydrants were frozen.

Although the pressure was increased from 46 to 98 lbs. per square inch, no serious trouble has been experienced with the old plumbing.

Service pipes are of "3A" lead and are required to have not less than a 5-ft. covering, and to have a valve and box at the curb.

Skaneateles water for manufacturing purposes and domestic use is strikingly superior to that furnished formerly from Onondaga Creek. This is obvious when a comparison is made between the two. In the report submitted to the Water Board in 1889, the following will be found as an average of the several analyses :

	Parts per 100 000.	
	Skaneateles Lake water.	Onondaga Creek water.
Solids, volatile	1.600	6.500
" fixed	11.433	48 350
Hardness, temporary	7.95	16.95
" permanent	1.80	19.46
Sulphates	0.497	20.778
Nitrates	0.023	0.059½
Chlorine	0.353	1.850

In the old supply, boilers became heavily encrusted with a hard scale which had to be removed by chemicals. Since Skaneateles water has been introduced, manufacturers say that the old scale on boilers has become decomposed and is now found in the form of mud easily disposed of by flushing with water.

For laundry purposes the water is excellent, many preferring it to

rain water that has been stored in cisterns, as it leaves articles washed in it whiter than rain water does. The additional amount of soap required to make suds, is hardly noticeable in ordinary family use.

As a potable water it is equal to any city supply in the country. In the report of 1889, Charles G. Currier, M.D., of New York City, says of it: "This water compares favorably with the best waters of which bacteriological analyses have been made." The small amount of mineral matter which it contains relieves it of that insipid taste which perfectly soft water has.

In conclusion, it affords the author pleasure to acknowledge the service rendered in the construction of these works by his assistants: Mr. F. M. Wakefield, who made the preliminary and location surveys; Mr. O. H. Bogardus, recently deceased, who was in charge of all work at Skaneateles Lake; Mr. R. J. Marcher, on Section No. 1 of the conduit line; Mr. William Kelley, Sections Nos. 2 and 3, and city work; Mr. C. A. Beach, Section No. 4, and the Woodland Reservoir; Mr. C. F. Taylor, city distribution; Messrs. W. S. Farrington and Thomas McE. Vickers, in charge of the drafting department; and other assistants.

DISCUSSION.

Mr. Croes. J. J. R. Croes, M. Am. Soc. C. E., had listened with great interest to the very clear and full description of the works recently constructed for supplying water to the important city of Syracuse, the population of which has grown from 51 792 in 1880 to 88 143 in 1890, and was believed to be over 100 000 at the present time.

Leaving to others the discussion of the engineering features of the paper, it seemed to him that a somewhat fuller account of the history of the works might be of interest as showing how a very important enterprise might be carried to a successful completion in spite of determined opposition by careful attention to the principle of first being sure you are right and then going ahead in a systematic and determined manner.

At the time the preliminary examinations to which the author has alluded were made in 1888, the city of Syracuse had been supplied for 60 years by a private company. The water furnished by this company was not palatable or wholesome, and the works generally were insufficient and in rather poor condition. The agitation for other supply had been in progress for some 20 years, and 17 years previous to this period the speaker had investigated the subject of a supply from the Tully Lakes under direction of the late Alfred W. Craven, Past President Am. Soc. C. E., and had reported on the same, and during the intervening period various projects had been broached and companies formed for the purpose of introducing a new supply from a variety of sources, so that when the speaker was called upon by the Water Commissioners in 1888 to investigate thoroughly the whole subject he found that there were 11 different sources of supply, each of which was considered by its advocates to be the best and only practicable one. These sources were the Salmon River, Skaneateles Lake, Lake Ontario, Seneca River, Onondaga Creek at Cardiff, ground water in the creek valley from driven wells, Cazenovia Lake, Oneida Lake, Otisco Lake, the Tully Lakes, and the Syracuse Water Company's works, comprising springs and the water of Onondaga Creek.

A very thorough examination, topographical, chemical and biological, was made of all these sources of supply, with the result that the speaker, as stated by the author, recommended the water of Skaneateles Lake as the best source, and presented such evidence in support of this recommendation that the Water Commissioners unanimously approved it, and after a bitter contention in the public press, at public meetings and in the Legislature, the citizens of Syracuse approved of the conclusion reached by a vote of 12 212 affirmative to 910 negative votes. This did not end the contest, however, for a prominent citizen of Syracuse and a member of this Society was still

so unconvinced and so impressed with the especial desirability of another source of supply that he brought a suit in the courts to restrain the city from undertaking the works on the ground that the law was unconstitutional. On June 15th, 1890, a special term of the Supreme Court confirmed the constitutionality of the law. On April 1st, 1891, a general term of the Supreme Court reversed that decision and on June 25th, 1891, the Court of Appeals reversed in turn the decision of the general term, and on December 22d, 1891, the Court of Appeals again decided that the law was constitutional and that the city of Syracuse could proceed to take the waters of Skaneateles Lake. Mr. Croes.

The principal ground of opposition to the scheme proposed was that the state of New York required the waters of Skaneateles Lake for the Erie Canal, and the contest in the Legislature, before the Canal Board, and before the Commissioner of Public Works and the State Engineer was chiefly promoted by the friends and adherents of the Erie Canal, including the Produce Exchange of New York City and the Board of Trade of Buffalo.

The law, as finally passed, required that the city of Syracuse should make good to the state of New York a supply of water equivalent to that taken from Skaneateles Lake. The method of compensating the state was the subject of a protracted contest in which the speaker took an active part, with the result that the counsel for the Water Board at the close of the contest kindly wrote: "You furnished the ammunition and we lawyers fired it off; the force of the ammunition produced the result."

At the time of the termination of the speaker's connection with the Water Board on June 30th, 1891, the final decision of the courts was still in abeyance, but immediately upon its being rendered, the Commissioners entered upon the work with the results which have been so clearly stated by the author.

To the speaker, some of the facts stated in the paper were extremely gratifying. The cost of the construction of the entire conduit line was estimated in the Report of 1889 at $747 141. The actual cost was $738 771.65. The value of the 36 water powers on the line of Skaneateles Creek was estimated in 1889 at $472 000. In the paper the entire cost is given as $462 000.

The value of the plant of the Syracuse Water Company which the Water Commissioners were authorized by the law to purchase was estimated by the speaker in 1889 at $850 000, and this was precisely the amount which the Commissioners of Appraisal awarded to the company in 1891.

At the time the preliminary estimates were made, the speaker considered it advisable to suggest cast-iron as the material for the conduit pipe. This was done mainly for reasons of policy. It was deemed safer to recommend the use of a well-known and long-used material for

Mr. Croes. the conduit line than to attempt to recommend the use of any novelties in construction, as they might have been considered at that time.

In April, 1891, the speaker in response to a request from the Water Commissioners, investigated the subject of pipe material with considerable care and then recommended that the conduit pipe in this particular instance should be made of plate steel under carefully prepared specifications and stringent inspection. It appears from the paper, however, that cast-iron was used, and it would be interesting to know the reasons which led to the adoption of that material, in view of the general sentiment which appears to prevail at the present time in favor of steel conduits.

Mr. Gould. E. Sherman Gould, M. Am. Soc. C. E.—Judging from the description and drawings of the dam at Skaneateles Lake, the masonry rests on a timber grillage, with an earthen embankment behind, and a single row of sheet piling underneath the masonry dam. This seemed to the speaker to be taking a good many chances on the foundation. Indeed, there is no foundation at all, as the timber platform or grillage appears to be placed on the surface of the ground. Setting aside the want of a proper depth to the foundation, he asked why, if the ground was solid, was a grillage needed at all, and if it were not solid, why was a timber platform, placed directly upon the surface, considered sufficient, with merely a row of sheet piling as a cut-off.

The speaker also criticised the design of the distributing reservoir, as lacking a center wall or core of masonry inside the embankment. The masonry facing is only or principally a protection against wash, and is always liable to crack and leak from settlement of the bank and other causes. It is true that in the case of a small distributing reservoir into which water is pumped, or into which it runs by gravity, as in the present case, the precaution of a masonry center wall is less needed, and the speaker's remarks applied more particularly to large earthen dams. It seemed to him that no lengthy argument could be needed to show how greatly the safety of an earthen dam is increased by the existence of a stout center wall of masonry, penetrating well into the sides of the valley and carried down to a proper depth below the surface. It raises the type of structure at once from one about which fears may be always entertained, to one which may be reasonably regarded as permanent. If water once begins to penetrate an earthen embankment, the gravest apprehensions should arise as to its safety. Such leaks may be and frequently are taken up by the silting in of material held in suspension, but it is quite certain that if an engineer should find water percolating through his earthen dam, he would be in a very uneasy frame of mind until the percolation was stopped. The reason is obvious. When water once finds its way through an earthen bank, the leak is very liable to go on increasing, because the bank is composed of an aggregation of minute particles readily carried

away by the water. If there is a center wall of masonry, however, small filtrations of water need not cause anything like the same degree of apprehension, because water can pass through crevices in the masonry almost indefinitely, without appreciably enlarging the passages. Mr. Gould.

The late William J. McAlpine, Hon. M. Am. Soc. C. E., in a paper read at the Annual Convention of 1889, at Seabright, gave three chief causes for the failure of dams :

First (and for two-thirds of the cases which he had examined), leakage of the outlet pipes or conduits.

Second, an incomplete connection of the natural and artificial earth at the bottom and sides of the valley, and

Third, the want of sufficient wasteways, leading in floods to the overflowing and consequent destruction of the dam.

Every one of these causes the speaker believed to be eliminated, or at least greatly reduced, by the presence of a good masonry center wall. A perfectly tight connection can be made with all the pipes or conduits leading through the bank, rendering it impossible for water to follow along their outer sides; any imperfect bonding of the embankment with the bottom and sides of the valley is remedied by the presence of a solid cut-off which effectually breaks the joint between the two, and should the dam be over-topped by a flood, its destruction would be either wholly prevented or at the worst so greatly retarded, that the consequent disaster would be robbed of a great part of its terrors. While the greatest care should be bestowed upon the construction of the bank, even when a masonry center wall is used, it will always give the engineer a great deal of confidence in his work to recollect that he has a safeguard against all those imperfections which are apt, in spite of the greatest vigilance, to creep into the best work.

A puddle wall is a very feeble and delusive substitute for a masonry one. For one reason, it offers no security against the attacks of musk-rats and other burrowing animals.

The speaker would ask in conclusion whether, in view of the appalling list of failures which stands recorded against dams and reservoirs the world over, a record which it is believed is exceeded by that of no other class of engineering structures, and which is increased by every freshet and flood which occurs, hydraulic engineers should not take a firm and united stand in favor of types which ensure the greatest degree of safety.

J. J. R. Croes, M. Am. Soc. C. E.—The New York Legislature limited the size of the conduit to 30 ins. because at the given gradient a pipe of that diameter will deliver 15 000 000 galls. a day. A careful study of the quantity of water which it was desirable to introduce into the city of Syracuse led the speaker to the conclusion that a 15 000 000-gall. supply would meet the wants of the community for such a period Mr. Croes.

Mr. Croes. of time that the interest charges would just equal the first cost of the works. The Water Commissioners agreed that a provision for a supply for that period would be the most judicious financially and limited their proposed expenditure to the sum estimated as needed for that purpose, and the law was so framed that the objection could not be made that it was proposed to take an unlimited supply from Skaneateles Lake.

With regard to the construction of earthen dams, the speaker stated his belief that an impervious core wall could be obtained of other material than masonry which would be just as good; it must be made of the best material, however, and well placed. Many miles of embankments and dams are now standing in the United States in which the center is a wall of puddled earth and clay, combined in suitable proportions and placed properly, and such walls are as impervious as any of masonry can be. It is true that a reservoir can be constructed, like one recently built on Long Island, N. Y., which is no better than a filter-bed; this is not the fault of the method of construction, however, but of the material, which is unfit for reservoir banks and cannot be made thoroughly impervious to water. That the mode of construction was not at fault is evinced by the fact that the Ridgewood Reservoir of the Brooklyn Water-Works, built after the same general plan more than 30 years ago, has been tight ever since water was turned into it. The puddle wall in the center of the bank and on the slopes was made with good clay mixed with the proper amount of gravelly earth and well worked with spades. Another reservoir of the same description is in Central Park, New York; it was built in 1860 by General George S. Greene, Past President Am. Soc. C. E., and has been perfectly tight ever since. Another such at High Bridge, New York City, built in 1869, has never leaked.

Mr. Bogart. JOHN BOGART, M. Am. Soc. C. E.—The signing of the Syracuse water papers, one of the last official acts of the speaker during his term of office as State Engineer of New York, was the culmination of a series of long-continued discussions and contests. There was an apprehension in the minds of some people who had to do with the Erie Canal, that the taking away of the amount of water which could flow from Skaneateles Lake through a 30-in. pipe at the established gradient might cause a scarcity of water in a small portion of the canal fed partially from this lake, thus possibly delaying boats. The representatives of the city of Syracuse, particularly the engineer who made the designs for the new works, stated to the State Canal Board, of which the speaker was a member, that the storage capacity of the lake could be increased so as to secure to the canal all the water which was really needed during the season of navigation. Much of the opposition to the proposed works came from the village of Skaneateles and from the mill owners along Skaneateles Creek. During the season of navigation

the water in the feeder between the lake and the canal was used by the different mills; they also used water in the winter, although the canal does not want it then. The speaker was convinced at the time of the hearings on the case, that the pondage of the winter rainfall from the water-shed would supply the city of Syracuse with all the water that would pass through the proposed 30-in. pipe. Therefore putting aside the question of mill rights along the creek, the canal could have all the water it ever had, and the city of Syracuse could have the surplus amount stored by raising the level of the lake, which was formerly allowed to run away during the season of non-navigation. Mr. Bogart.

Concerning the matter of reservoir embankments, the speaker did not agree with Mr. Gould's remarks to the effect that a dam or water-bearing embankment built of earthy materials must be made absolutely impervious, for if water were to percolate through it, then failure would result. On the contrary, he knew from personal experience with many miles of earthen embankments which have been leaking for a long time, that many of them have been silting up gradually instead of giving away. These banks were built entirely of earth, generally with a puddle core. The failures which occur so often have seldom been due to the percolation of water through the earth, but a constant cause of failure has been the location of a pipe or something of that nature in the bank. This should not be done except with extreme precautions.

An embankment of sand and some puddle was instanced which leaked very much at first; the water was a little muddy and gradually silted up the bank. The speaker did not wish to go on record as being opposed to a masonry core, and he had under consideration at that moment a dam in which he was strongly inclined to put one. On the other hand, he regarded such a core as an extreme precaution for the sake of safety. If he had just the material wanted and was certain that water would not rise to the top of the dam he was designing, he might be willing to build it without any masonry core, although it is 60 ft. high. In the construction of all earth dams, however, it is absolutely necessary to make provisions for the safe flow from the reservoir of as much water as can ever enter it. In the discussion in Volume X of the *Transactions*, Joseph P. Davis, M. Am. Soc. C. E., states:

"In the construction of the dams for the storage reservoirs on the Sudbury River, the overflow weirs were proportioned to carry off without injury, floods of 160 cu. ft. per second per square mile. Although the gaugings had not shown a discharge of more than one-fourth of that amount, the examinations of Mr. James B. Francis proved that such a flood was probable at some time."

The rule there adopted seemed to the speaker to suggest the principle which should be followed by every engineer who designs a storage reservoir. Fairly accurate statistics of rainfall can generally be secured, and frequently there is to be found some information as to

Mr. Bogart. particularly heavy storms in the region of the water-shed. If suitable provision is made for wasting all the surplus water which can by any possibility reach the reservoir, there will be no danger of the dam being overtopped by the water behind it.

The author states that "to meet the requirements in the law authorizing the city to take water from Skaneateles Lake it was necessary to increase the storage capacity of the lake sufficiently to store therein the ordinary flow of its water-shed." This additional storage capacity was obtained by building a dam higher than the old dam, thus raising the maximum water level of the lake. In the arguments preceding the authorization of the works, Mr. Croes, as the engineer representing the city of Syracuse, suggested that the available storage capacity of Skaneateles Lake might be just as well increased by lowering the outlet from which water was taken, thus enabling a large volume to be drawn off which could not be taken under the conditions existing then. But there was one member of the Canal Board, whose consent was required before work could begin, who could not see the matter in that light, so the level of the lake had to be raised, involving considerable expense to the city.

Mr. Emery. CHARLES E. EMERY, M. Am. Soc. C. E.—There were other considerations than those mentioned by Mr. Croes which led to the final adoption of the 30-in. conduit recommended in the engineer's reports. The mill-owners had an interest in the water as it flowed over the dam at Skaneateles Lake to supply the Erie Canal, and in all the legal arguments a pipe of this size was referred to as limiting the quantity which could be taken, its capacity exceeding but a trifle the amount of water condemned. In these suits it was considered important that as much water should go into the Erie Canal after as before the construction of the proposed water-works, so that the mills should not be deprived of their water powers. It was also contended that the mills should be reimbursed for the loss of power during the winter when the lake was filling and no water flowed from it. In these suits of a mixed engineering and legal character, interesting technical questions are often involved. In this case the contention was principally as to the value of the water which would be lost to the mills while the lake was rising, since during the rest of the year there would be no alteration of the old conditions. The city's defence was that very little water flowed through the canal feeder in the winter. At one of the mills where a large quantity of water was used in washing woolens, the city claimed that the water-shed between the lake and mill was sufficient to furnish the supply needed. The answer to this was that the slope was so steep and the ground so hard that the rainfall ran off in little freshets, and that the rocky bottom of the stream was so porous that part of the water passed under the woolen works by subterranean channels. The latter contention was demonstrated forcibly by placing

some aniline dye in the creek above the establishment, which disappeared and came to the surface again below the works. Mr. Emery.

The abstract value of the water, apart from the power which it might furnish, was brought up in the Syracuse cases, but the most interesting decision on this subject is that relating to the claim of the American Print Works of Fall River. There was no trouble in agreeing upon the flow of the stream from which this plant took water. The company only owned a fraction of the current flow for a fraction of the total fall, and the condemnation was but a small percentage of the whole, but the appraisers awarded this company $10 000 because the water was used for washing, etc., and nothing else could replace it, while a less sum was divided among a considerable number of owners of water power rights on the stream. Afterward another condemnation was agreed to on the same basis of compensation, but when it became evident that still another condemnation must be made it was suggested that the owners allow the city to take all the water required on consideration that the city remit the taxes of the mills for water power, it being evident that as the use by the city increased there would be less water power to be taxed. The adjustment of the various interests was an interesting problem, and the speaker was finally able to demonstrate that the compromise was for the interest of both parties, by assuming a water consumption in the future based on the probable increase in population and comparing the taxes on the value of the water with the interest on the bonds which the city would be obliged to issue to pay for the condemnations. Eventually the compromise was agreed to by both parties.

J. N. Greene, M. Am. Soc. C. E.—While the speaker had never built a dam with a masonry core wall, he considered them an extra safeguard. The banks he had constructed had a central puddle core, the puddle being of the best gravel and clay, without any loam. With such material, properly placed and thoroughly packed, a dam can be made impervious to water. The objection to the masonry core wall is that it is seldom tight as usually constructed. The water penetrates through the earth to the masonry so that eventually the entire up-stream face of the core is exposed to a hydrostatic pressure, and the water passes through the masonry into the lower side of the dam. To obviate this and to combine all the advantages of puddle and masonry in one structure, he recommended building a masonry core and placing a puddle wall against its upstream face. Mr. Greene.

Many earth dams have gone out because engineers have only made them impervious up to the assumed high-water line. During unusual freshets the water would rise above this line, where there was no puddle or core to prevent its percolation through the earth. The top of the dam would be washed out and then its face, resulting in the partial or complete failure of the entire structure. Mr. Bogart's

Mr. Greene. statement that the water will not overtop the bank when the spillway or other outlet is suitably designed the speaker considered true except on extraordinary occasions. The latter do occur, however, and provision must be made for them by making the dam impervious to its top.

Mr Buck. L. L. Buck, M. Am. Soc. C. E.—The nature of the material on which the dam or embankment is to be built must be considered in determining the material and form of construction of the dam itself. If the dam is to be built on solid rock, a masonry core might be useful in some cases, but even on a rock foundation a puddle wall has been found to serve the purpose fully. But in works like the Mississippi levees the masonry core would be an exceedingly doubtful construction. The same conditions frequently occur in canal embankment. As for the insertion of pipes through a reservoir embankment, it is not to be supposed that an engineer would lay them directly in the earth. If he could not build a gate-house for the pipes, he would at least have a series of flanges around the outside of the pipe so as to bond thoroughly with the puddle wall.

Mr. Christian. G. L. Christian, Assoc. M. Am. Soc. C. E.—The paper states that the cement was tested in the form of a mortar mixed in the proportion by weight of 1 part of sand and 1 part of cement. The general method of testing cement throughout the country, the speaker believed, is in a mortar in which the sand and cement used are proportioned by volume, not by weight, and he thought the author should explain the reason for his selection of another method.

CORRESPONDENCE.

J. L. Van Ornum, Assoc. M. Am. Soc. C. E.—Engineers will be interested in a fuller explanation of the surveys, the accuracy of which has so great an influence on economy of construction where alignment and grade are most important factors, and the avoidance of any amount of rock excavation means a considerable decrease in cost. The author states that the topographical notes were obtained by sighting along the edge of a slope board so graduated that a suspended pendulum indicated the horizontal distance between contours; knowing the length of the slope, the number of contours on the slope is known and plotted. Will the author state how the length of the slope was measured? The method of graduating the board indicates that it was measured horizontally. From the fact that a contoured map was plotted from these observations, it is evident that such distances were accurately obtained. A contoured map does generally and should always involve the principle of exactness within the limits of its scale. Engineers are coming to use instruments of precision for obtaining topographical data, with little or no extra expense. If the slope board, formerly so much used on railroad topography, still holds a field of peculiar applicability, they will be interested in having that field defined. Mr. VanOrnum

The writer believes the care and ability shown in the laying and inspection of the pipe line to be a valuable example, especially the hydraulic test on the pipe in position before the different sections of the trench were finally refilled.

John W. Hill, M. Am. Soc. C. E.—The bacterial condition of the water of Skaneateles Lake, according to Dr. Currier's report, is remarkable, and except sewage contamination of the water occurs at some time in the future, the typhoid rates for Syracuse, so far as the local water supply is concerned, should be very low. The reference to living bacteria is unnecessary, the dead bacteria are of course beyond the power of reproduction, and even if pathogenic varieties were among them, they are beyond the possibility of harm. It is very gratifying to know that in passing on the quality of this water, chemical tests alone were not relied upon. Heretofore the chemical test has been regarded as sufficient for sanitary purposes, when it is well known that some of the most harmful of pathogenic germs will flourish for a long time in water, which from the chemical point of view would be eminently potable. Mr. Hill.

It is suggested that a comparison of the typhoid fever rates for the five years previous to the introduction of the Skaneateles water, and for a few years after will furnish valuable information to those of the profession interested in the purely sanitary features of water supply,

Mr. Hill. for the true test of the quality of the water will be its influence on the health of the citizens of Syracuse.

Mr. Le Conte. L. J. Le Conte, M. Am. Soc. C. E.—The author pictures the usual malady which kills water companies throughout the country, namely, their inability to anticipate the rapid growth in population, as well as a general lack of judgment and foresight in grasping the irresistible necessities which the future always develops. In all cases the public is the body to be satisfied, and they must be gratified *nolens volens*.

The 30-in. cast-iron conduit is made of heavy iron, and this fact suggests the propriety of using steel pipe instead. The writer has no doubt that the item of transportation and hauling these heavy castings over such a rough broken country to the site of the pipe line must have been a slow and expensive operation.

The 4-in. air valves, as shown, are automatic only when acting as vacuum valves, while the pipe line is in process of being emptied, and to let air escape from the pipe line the valve has to be forced open by means of a screw stem overhead. Looking at the profile, Fig. 10, it is natural to suppose that at the light-pressure summits, say at 4.5, 12.0 and 15.5 miles. there will be a constant tendency toward the gradual accumulation of air, and to such an extent as to seriously retard the flow of the water column unless removed from time to time.

The air valves used on the Pacific Coast nearly all work automatically both ways, that is to say, they will allow confined air to escape, and also will act as a vacuum valve and allow air to enter the pipe line while being emptied. This double automatic action is best accomplished by use of a heavy ball-float immediately below the valve seat. By its flotation the valve is closed up against its seat, and by its weight the valve is pulled down and open when air accumulates.

Cement-lined pipe has been used largely in California for the past 30 years. Experience seems to show that so far as durability is concerned, the larger sizes, 14 to 16 ins. in diameter, used as street mains, which suffer little or no shocks from water hammer, have lasted very well. On the other hand, in the smaller sizes, near hydrants under high pressure or near elevator valves, where they are subjected to sharp periodic water-hammer shocks, it is found that the inside lining crumbles all to pieces in a short time. Watering carts and steam fire engines have been often choked up and disabled by the débris discharged from the hydrants during a heavy draft on the street main. Hence this class of pipe is being taken up and replaced with cast-iron for distribution systems.

Mr. Hill. W. R. Hill, M. Am. Soc. C. E.—Replying to Mr. Croes, the author would state that as the Legislature had limited the size of the conduit to 30 ins. in diameter, it was important that it should be of such mate-

rial and so constructed that it would carry the greatest possible quantity of water. With a steel pipe the flow of water would be retarded by the seams and rivet heads. Emil Kuichling, M. Am. Soc. C. E., Chief Engineer of the Rochester Water-Works, made a very thorough examination of the loss of velocity due to seams and rivet heads in a 24-in. wrought-iron conduit and found the loss to be fully 10% as compared with a cast-iron conduit of the same diameter. Mr. Hill.

In answer to Mr. Gould, the author would say that the grillage under the dam at Skaneateles Lake rests on a very compact bed of stiff clay. Around the timbers the material was thorough-puddled and many timbers were bedded in concrete with the space between them filled with the same material. On the up-stream side of the grillage at the bulkhead a row of sheet piling was driven. Along the top of this the material was excavated to a depth of 2 ft. This space was filled with concrete to a height of 1 ft. above the tops of the piles, completely covering them. On the down-stream side of the grillage a frame of timber faced with plank was set in a trench 4 ft. deep, filled with puddle. The bed of the stream below the grillage was paved with stone. Through the center of the dam a row of 5-in. sheet piling was driven to a depth of 12 ft. Between this row and the down-stream edge of the grillage, another row of piles was driven to a depth of 8 ft. The author has no fear of any stream of considerable size finding its way under the dam through the piling; in fact the gates of the dam were closed during the entire winter of 1894 and 1895, and there was scarcely any leakage. A grillage in this case was adopted for its desirable use as a flooring at the bulkhead, for a foundation of the cast-iron sluices, and for its use as an apron below the gates and spillway. It will be noticed that the spillway extends nearly the length of the dam.

Referring to Mr. Gould's remarks relating to the reservoir embankment, the author thought that if hydraulic engineers find a suitable location for an earthen dam with a firm foundation of hardpan or other impervious material and with good material of clay and gravel to make the embankment, they should not needlessly expend the money of a municipality or corporation in constructing a masonry core wall. Earthen dams, like all others, should be built with the greatest possible care. The embankment of Woodland Reservoir was built and allowed to settle one year before it was faced with concrete and paving.

Replying to Mr. Christian, the author is not fully satisfied that the general method of testing cement throughout the country is a mortar in which the sand and cement used are proportioned by volume, not by weight. The United States engineers in their specifications for the important work of constructing the Sault Ste. Marie lock required that the proportions should be obtained by weight, and many others do the same.

Mr. Hill. Replying to Mr. Van Ornum as to how the length of the slope was measured in taking topographical notes, the author would state that where great accuracy is required, it would be well to measure the horizontal length of the slope, but for ordinary work the distance could be estimated.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

756.

Vol. XXXIV.—August, 1895.

THE TEMPERATURE OF LAKES.

By DESMOND FITZGERALD, M. Am. Soc. C. E.

READ AT THE ANNUAL CONVENTION, JUNE, 1895.

WITH DISCUSSION.

For several years past the author has published in his annual reports to the Boston Water Board the monthly temperatures at the surface, mid-depth and bottom of Lake Cochituate, the storage basins and distributing reservoir connected with the Boston Water Supply, together with the temperatures in the city mains. These tables are based on weekly observations made at the time of collecting samples for analysis, and as the temperature is not the principal object, and dependence is necessarily placed upon unskilled observers, the results cannot always be depended upon to the tenth of a degree, although they are so expressed. The observation of the correct temperature of water is a more difficult matter than is generally realized. Especially is this true at the bottom of a deep lake where it takes more than an hour to get the reading with mercurial thermometers to within a tenth

of a degree. With the thermophone a more accurate result is now obtained in a few minutes.

M. de Saussure, who was the first to take the temperature of deep lakes, used a thermometer protected against pressure and surrounded with non-conducting substances, cloth and wax, which allowed the instrument to be hauled up 1 000 ft. without changing the reading 0.1° Cent. The deep sea thermometers of Negretti and Zambra and of Cassela have very much facilitated the taking of bottom temperatures, but the thermophone as developed* and first used on the Boston Water-Works has revolutionized the methods of observing temperatures in the author's practice. This instrument allows the temperatures to be read at the surface by aid of the telephone. It depends upon the principle of the Wheatstone bridge. An electric circuit is formed through two small wires of different metals, which are enclosed in a metallic case sunk to any desired depth. The other two arms of the bridge are formed at the surface, the telephone taking the place of the galvanometer. This instrument is so accurate that its results can be depended upon to much less than 0.1° Fahr., and a series of temperatures throughout the vertical can be taken with an allowance of about a minute for each point observed.

As the old tables, already referred to, contain so many inaccuracies, it has not been thought worth while to print them in full in this paper, but the averages are given in Table No. 1. The surface temperatures are generally not far from right. The deeper the level, the greater the inaccuracy as a rule. For the bottom temperatures in Lake Cochituate the observer at times kept his bottle and thermometer at the bottom and pulled them up only for record. By studying this five-year series in connection with the thermophone tables herewith printed, the author has been able to form an accurate idea of the seasonal changes going on in large bodies of water.

Surface Temperatures.—By combining the surface temperatures taken at any particular spot for five years, a mean has been obtained. These means are so similar that they have all been combined in one diagram, which is believed to represent approximately the surface temperature of bodies of water of depths varying from 20 ft. to 80 ft. and under the same climatic conditions as those which prevail in the vicinity of Boston; but it must be borne in mind that five years is rather a short period

* By Messrs. H. E. Warren and G. C. Whipple.

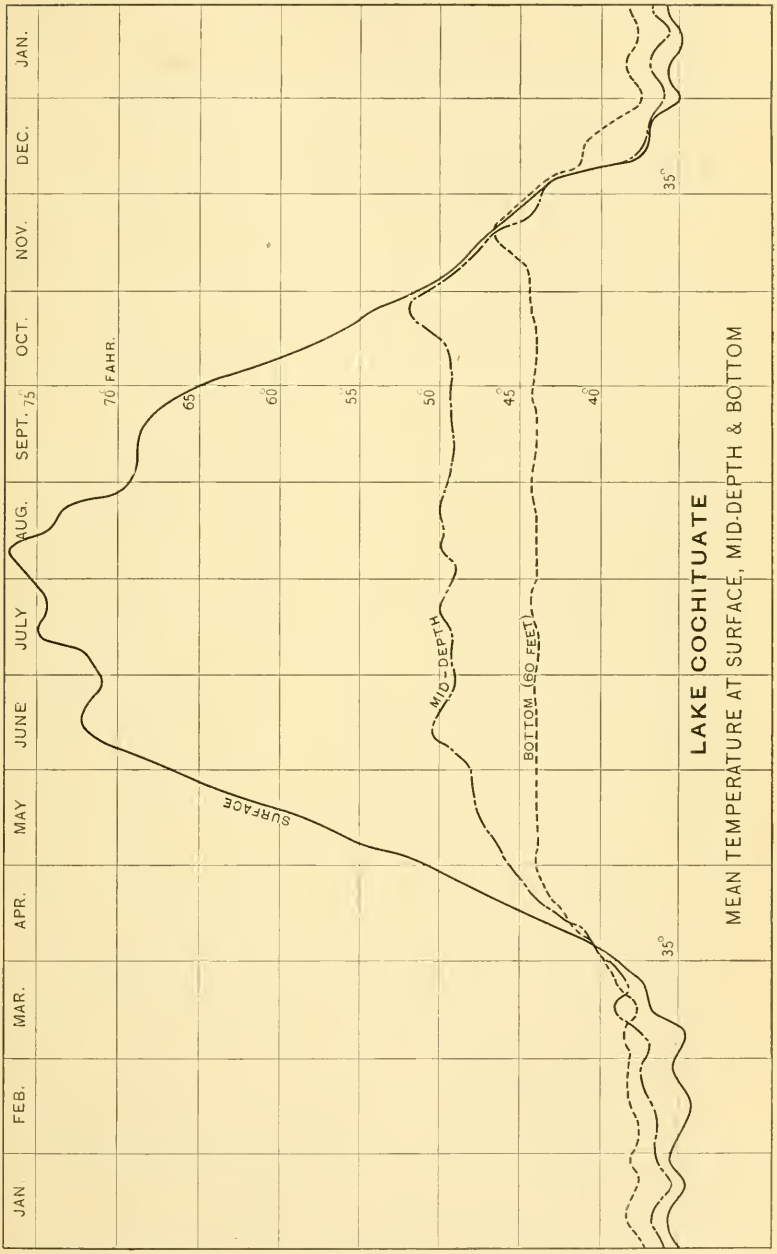

JAN
FEB.
MAR.
APR.
MAY
JUNE
JULY
AUG.
SEPT.
OCT.
NOV.
DEC.
JAN.
75° FAHR.
70°
65
60°
55
50°
45°
40°
35°
SURFACE
MID-DEPTH
BOTTOM (60 FEET)
LAKE COCHITUATE
MEAN TEMPERATURE AT SURFACE, MID-DEPTH & BOTTOM

FIG. 1.

from which to form a mean, and that some of the irregularities might be smoothed out and somewhat different values obtained by a longer term of observations. This diagram is not given here for the reason that it is quite similar to that given for Lake Cochituate in Fig. 1.

During the winter the water under the ice is sufficiently near the freezing point to fix it at 32° Fahr. from the latter part of December to the breaking up of the ice in the spring. The surface then begins to warm up at a uniform rate from 40° in the latter part of March or first of April, to 72° in the middle of June, making the mean temperature for April about 45°, and for May about 58°. In the middle of June there is a noticeable depression in the curve, which rises to but 73° on the first of July; in the Cochituate curve it actually falls to 71°. The temperature then rises sharply to 75° by the middle of July on both curves. The mean temperature for June is approximately 71°. In the middle of July there is another depression in both curves, and the temperature is nearly uniform (75°) during the remainder of the month, leaving the average temperature for July at about 74°. The curve rises slightly in the first week in August to 76.5°, which is the summit of the curve, and from this time the temperature falls rapidly to about 69° on September 1st. The mean temperature of August is, therefore, about the same as that for July. After the first of September there is a very interesting flattening of the curve until the middle of the month, when it is still at 69°. Then there is a drop to 65° on October 1st. The mean temperature for September is about 67.5°. Throughout October the curve is almost a straight line, reaching 50° by the mean curve and 51.5° by the Cochituate curve on November 1st. The mean for October is about 57°. The cooling goes on rather more slowly in November, reaching 40° on December 1st in the mean curve, and 44° on the Cochituate curve. The mean temperature for November may therefore be set down as somewhere in the neighborhood of 46°. By the middle of December the temperature reaches 37°, and after this time it drops quickly to the freezing point if in a shallow pond, and more slowly if in a deep one.

This heating of the surface of a body of fresh water during the larger part of the year gives rise to some interesting phenomena in connection with the subject of water temperatures and also with questions affecting a domestic water supply, as will be shown.

Bottom Temperatures.—If a pond is less than about 25 ft. in depth,

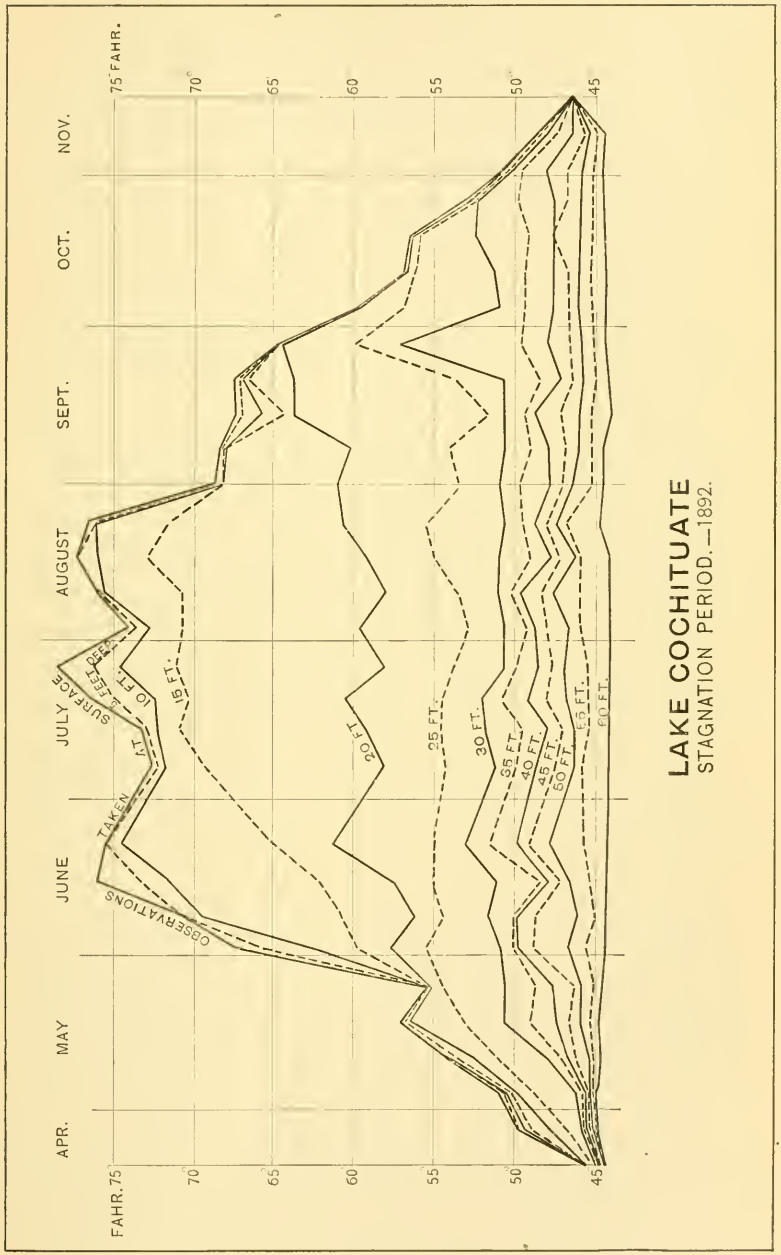

LAKE COCHITUATE
STAGNATION PERIOD.—1892.
APR.
MAY
JUNE
JULY
AUGUST
SEPT.
OCT.
NOV.
FAHR. 75°
70°
65°
60°
55°
50°
45°
OBSERVATIONS
TAKEN
AT
SURFACE
2 FEET DEEP
10 FT.
15 FT.
20 FT.
25 FT.
30 FT.
35 FT.
40 FT.
45 FT.
50 FT.
55 FT.
60 FT.

FIG. 2.

the bottom temperature does not vary very much from that at the surface, although in the winter it may be 5 or 6° warmer, and in the middle of summer as many degrees cooler. These shallow ponds or lakes do not provide the interesting phenomena which are to be discussed, and but little more attention will be given to them. The winds in summer keep the water stirred up throughout, and there is little stagnation of the bottom layers, except, perhaps, for a short time in the winter.

With deeper lakes, however, a very different state of things prevails. Starting with the freezing over of a deep lake about January 1st, the bottom temperatures, as they exist on the Boston Water-Works, will be now traced. The point of maximum density of fresh water is 39.2°. When Lake Cochituate freezes, the temperature of the bottom is generally near this point in the deep portions, although in a lake of the same depth, say 70 ft., the temperature may be much below this if the weather is severe and the winds high before the ice forms. The several strata lie in their order of density, decreasing gradually until within a few feet of the surface, when they suddenly fall to the freezing point adjoining the ice. The form of this curve is quite persistent in all lakes, although there are many variations from it in places where there are currents or other local modifying causes, as may be seen by an inspection of the tables and diagrams in this paper. In a general way the whole body of water remains unchanged after the ice forms until it is broken up in the spring, but there are some interesting details connected with certain changes going on even under the ice which will be discussed later. It is desirable first to obtain a general comprehension of the subject.

As the surface warms about April 1st to the same temperature as the bottom, the whole body of water is in unstable equilibrium, and circulation takes place from top to bottom under the action of the winds and currents. Finally, by the first of May, when the surface temperature has risen and gained about 5° upon the bottom, the difference in density seems to be sufficient to prevent any further warming of the bottom; and while the surface keeps on warming, the bottom remains at almost exactly the same temperature throughout the long period of stagnation until the middle of November, covering a period of nearly seven months, and varying during that time, so far as can now be determined, only a few tenths of a degree. Of course, this

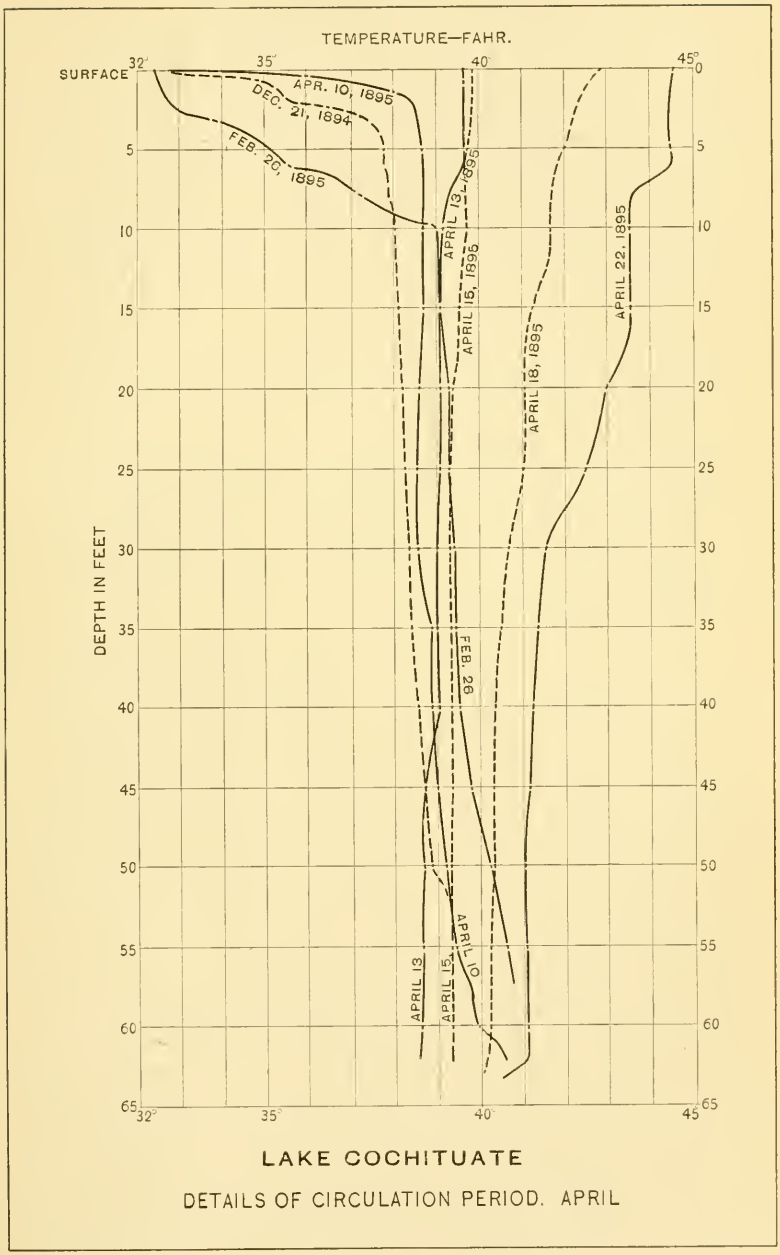

TEMPERATURE—FAHR.
SURFACE
32°
35°
40°
45°
APR. 10, 1895
DEC. 21, 1894
FEB. 26, 1895
APRIL 13, 1895
APRIL 15, 1895
APRIL 18, 1895
APRIL 22, 1895
FEB. 26
APRIL 13
APRIL 15
APRIL 01
DEPTH IN FEET
0
5
10
15
20
25
30
35
40
45
50
55
60
65
LAKE COCHITUATE
DETAILS OF CIRCULATION PERIOD. APRIL

Fig. 3.

means that the winds do not penetrate to this depth; neither are there convectional currents to change the temperature, and, as water is a poor conductor of heat, the sun's rays, however hot, are practically powerless at a depth of 65 ft. in Lake Cochituate. It will be noticed in Fig. 1 that the bottom temperature begins to rise in the middle of March from about the point of maximum density, but that it falls slightly before rising. This was thought to be an error of observation, but careful investigation with the thermophone during the present year has shown this not to be the case. The water before heating at the bottom first cools slightly. From the middle of March to the first of May the temperature rises to 44°, and the early part of April may be called the first period of circulation, the water for a short time remaining at the same temperature from top to bottom. From May 1st to November 1st the bottom temperature remains at 44°, and rises between November 1st and November 15th about 2° or 3° to meet the surface temperature, and then the whole mass of water falls at nearly the same rate in a second great period of circulation until freezing begins, which separates the surface and bottom, the latter remaining somewhat nearer the point of maximum density.

Turning for a moment to the other reservoirs, their bottom temperatures will be found to follow those of Lake Cochituate, but with this difference. As Basins 2, 3 and 4 are shallower, the winds are able to keep up more of a circulation, with the final result of less difference of temperature between the top and bottom layers, so that in the case of Basin 4, in a depth of about 50 ft. of water, the permanent summer temperature of the bottom is not reached until about a month later than in Lake Cochituate, and it is then about 9° higher, or 53°. Basins 2 and 3 are not deep enough for their bottoms to remain stagnant, but their temperatures are proportional to their depths.

Summer Stagnation Period.—For four years, 1891–94, the temperatures in Lake Cochituate have been taken weekly for every 5 ft. in depth during the summer stagnation period, April to November. These results are found in Table No. 2, and those for 1892 have been plotted in Fig. 2. The diagram enables the effects of the wind on the different layers to be studied. It will be noticed that throughout the period the temperature at a depth of 10 ft. is about the same as at the surface, showing that the winds keep the water stirred up thoroughly to this depth, but below 15 ft. the effect of the wind is

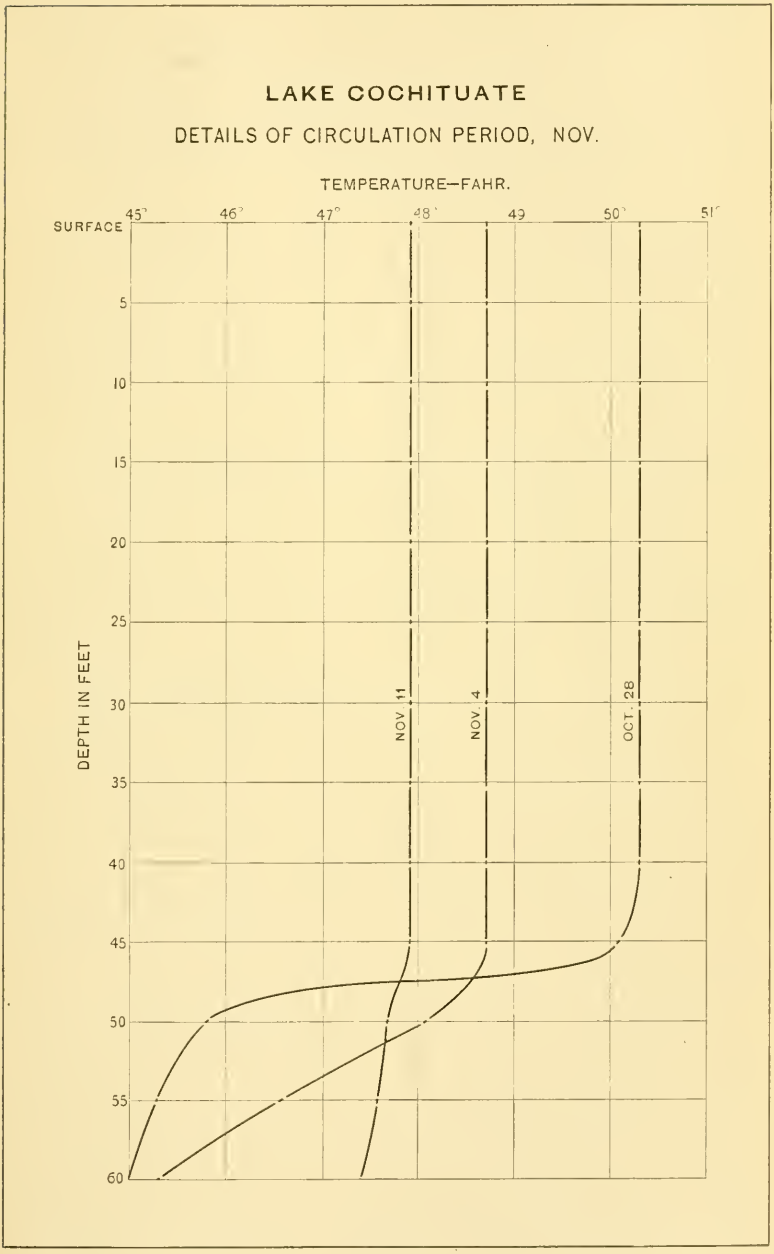

LAKE COCHITUATE
DETAILS OF CIRCULATION PERIOD, NOV.
TEMPERATURE—FAHR.
45°
46°
47°
48°
49
50°
51°
SURFACE
5
10
15
20
25
30
35
40
45
50
55
60
DEPTH IN FEET
NOV. 11
NOV. 4
OCT. 28

Fig. 4.

small. It must be remembered, however, that a season of the year is now under consideration when the difference in density is greatest between the top and bottom, and it will be found later that when this density is less, the effect of the wind reaches very much lower. As the layers are followed to the bottom, it will be noticed that they warm up less and less, until near the bottom the temperature lines are parallel.

Effects of Stagnation.—Some of the effects of stagnation are very remarkable. They have been fully set forth by the author in the Boston Water Board reports, and it will be unnecessary to repeat them here. It may be said, however, that in a lake with any considerable amount of organic matter in it and also in deep artificial storage reservoirs, where the surface has not been stripped, the lower layers, which are quiescent during the great stagnation period, gradually collect all the organic matter from the upper layers, and decay goes on until the oxygen is used up. The water becomes darker and darker, until by October it is very yellow, and generally has a disagreeable smell. Of course, when the great overturning comes, in November, all this bad water is brought to the surface, and the infusoria and diatoms begin to grow in enormous numbers, because the organic matter and oxygen are brought together and provide food for organic life. The same phenomenon takes place in the spring period of circulation, although on a smaller scale.

It is astonishing to a novice to take him out in a boat when the surface temperature is near 80°, and sink a bottle and bring up water icy cold from the bottom. In this connection it is interesting to consider that if Lake Cochituate were 1 000 ft. deep instead of 65 ft., this water from the bottom would probably be only 4° colder than it is. It is also possible toward the end of the stagnation period, with a little knowledge of a lake, to draw water of any desired quality according to the layers selected.

At the surface the water may be, and generally is, free from color and of excellent quality, while that at the bottom is in the worst condition. In a lake free from organic matter or in one which is so large that the organic matter brought in by the feeders is completely oxidized, or in a large artificial storage reservoir thoroughly constructed from a sanitary point of view, the phenomenon which has been described is almost entirely lacking, but it is somewhat rare to find a lake of such great purity that the water is as good at the bottom in

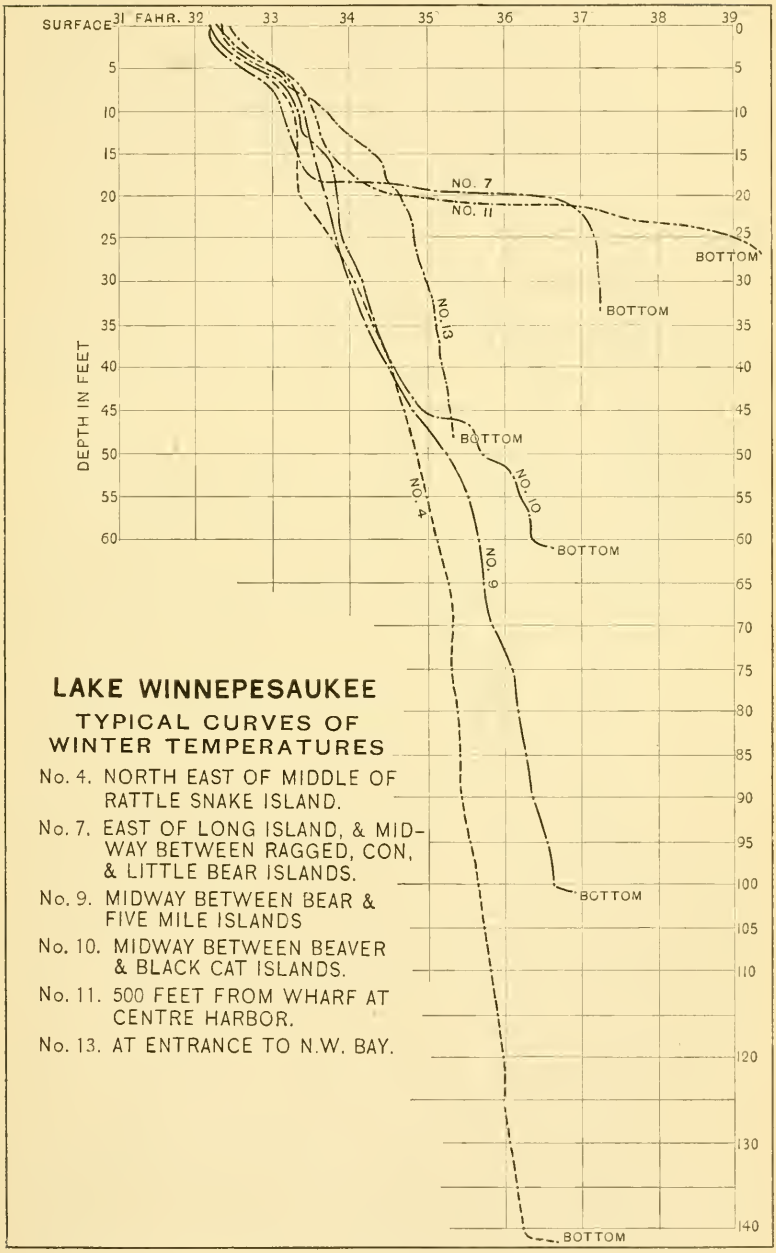

SURFACE
31 FAHR. 32
33
34
35
36
37
38
39
DEPTH IN FEET
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
75
80
85
90
95
100
105
110
120
130
140
NO. 7
NO. 11
BOTTOM
BOTTOM
NO. 13
BOTTOM
NO. 10
NO. 4
BOTTOM
NO. 9
BOTTOM
BOTTOM
LAKE WINNEPESAUKEE
TYPICAL CURVES OF WINTER TEMPERATURES
No. 4. NORTH EAST OF MIDDLE OF RATTLE SNAKE ISLAND.
No. 7. EAST OF LONG ISLAND, & MIDWAY BETWEEN RAGGED, CON, & LITTLE BEAR ISLANDS.
No. 9. MIDWAY BETWEEN BEAR & FIVE MILE ISLANDS
No. 10. MIDWAY BETWEEN BEAVER & BLACK CAT ISLANDS.
No. 11. 500 FEET FROM WHARF AT CENTRE HARBOR.
No. 13. AT ENTRANCE TO N.W. BAY.

FIG. 5.

the latter part of summer as it is at the surface. Experience in these matters will enable a fair opinion to be formed of the quality of a water without even a microscopical or chemical analysis, but this does not mean that a great many analyses are not necessary before one can judge of the quality of any water with scientific confidence.

A knowledge of the facts here narrated should lead to letting out or wasting the bottom water before November 1, when the construction of deep reservoirs will allow. In the case of deep artificial reservoirs containing organic matter, where the dam is at the lowest part of the valley, this is easily accomplished and should result in preventing the distribution of the impurities. A person living near a deep lake might arrange to draw good water from the surface through a long coil placed at the bottom and in this way secure good and very cold water all summer.

Circulation Periods.—The spring and autumn circulation periods already referred to, when the water is at the same temperature from top to bottom in the lakes under discussion, contain some interesting changes. In Table No. 3 and Fig. 3 will be found some accurate observations taken in Lake Cochituate under the ice from December 21st, 1894, to the breaking up of the ice on April 10th, 1895, and for several days after this last-named date.

It will be seen that on December 21st after the ice had formed, the temperature at the surface under the ice was at 32° and rose rapidly in the first 5 ft. to 37.8° and then very regularly to 39.2° at the bottom. On February 26th the temperature of the bottom had risen to 40.7° with some cooling effects in the upper layers, due probably to the emptying of a large mass of cold water into the lake from another source of supply, the Sudbury River. The heating of the lower layers may have been due to springs, the heat of the earth, to chemical action or to some cause which the author does not now comprehend. From February 26th to March 29th, there was very little change in the lower 40 ft. of water. On April 10th, when the ice was broken up into floating cakes and there was a very high wind, the surface was found at 33°. At a depth of 2 ft., it was at 38.4° ; at 6 ft., 38.7° ; and by tracing the curve below this point it will be seen that the lower 40 ft. of water had cooled about half a degree, although the bottom still remained at the same temperature as in February and March. On April 13th, the bottom had cooled to 38.6°, while the surface had warmed to

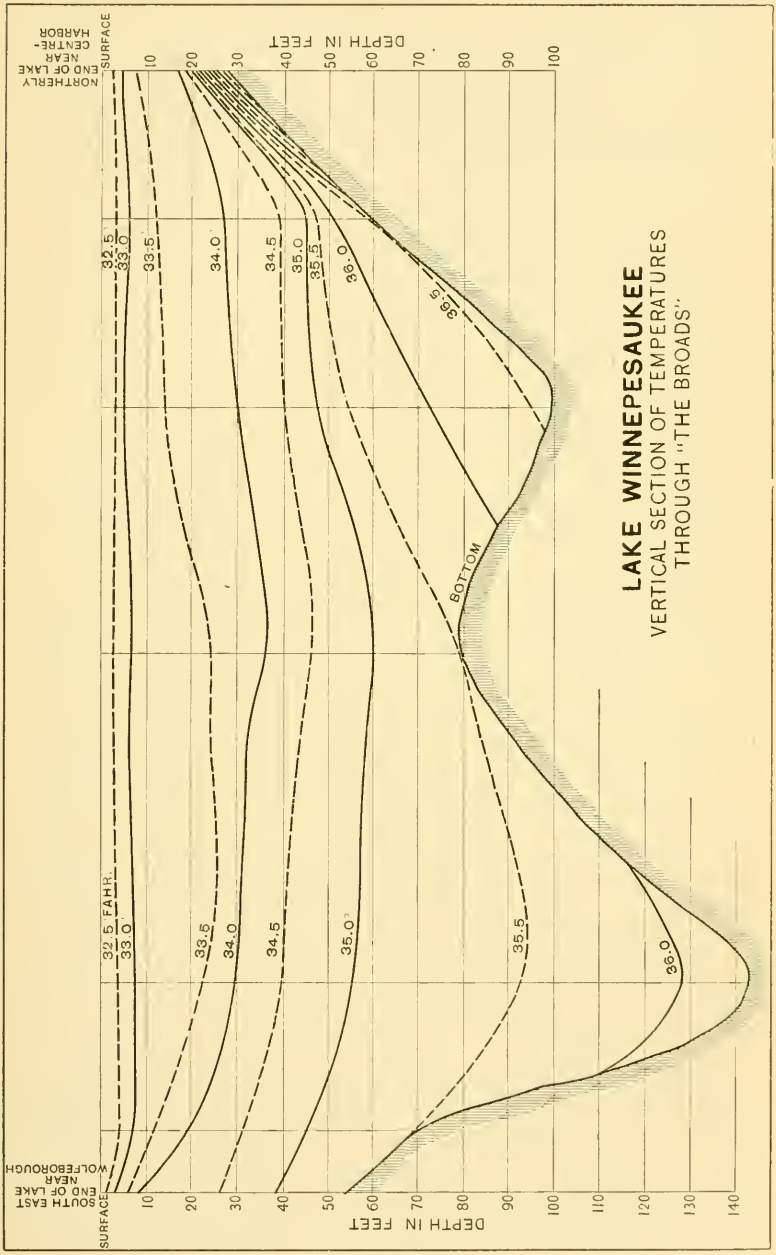

LAKE WINNEPESAUKEE
VERTICAL SECTION OF TEMPERATURES
THROUGH "THE BROADS"
SOUTH EAST END OF LAKE NEAR WOLFEBOROUGH
NORTHERLY END OF LAKE NEAR CENTRE-HARBOR
SURFACE
BOTTOM
DEPTH IN FEET
32.5 FAHR.
33.0
33.5
34.0
34.5
35.0
35.5
36.0

Fig. 6.

39.8°, and at a few feet below the surface and for a long distance toward the bottom, the water was very nearly at the point of maximum density. In other words, while the surface was warming up, the bottom was getting perceptibly colder, and it may be thus considered as proved that when the mass of the water is in this condition of unstable equilibrium, the wind and currents change the temperature rapidly from the surface to the bottom. These changes cannot be due to conduction, for it was shown that for half the year, while there may be 30° difference at times between the surface and the bottom, there is no appreciable heating of the bottom layers by conduction. From the observations taken after April 13th, it will be noticed that the heating of the whole mass of the water goes on until about the first of May, shown in Fig. 2, when there is a separation of the surface and bottom temperatures, as already described.

An examination of the tables shows that in the cooling of the lake in the autumn the reverse process takes place, and there is a warming of the bottom layers before they begin to cool. This is shown graphically on Fig. 4.

Lake Winnepesaukee.—Table No. 5 and Fig. 5 contain some observations taken under the direction of the author in Lake Winnepesaukee. This lake is situated to the north of Boston and contains 70 sq. miles of water surface. It is 150 ft. deep in the deepest portion, so far as ascertained. As the water of Lake Cochituate at 65 ft. depth was found at about 44° in summer, and the other reservoirs were found proportionately warmer as they decrease in depth, it was expected that Lake Winnepesaukee would be found colder at 150 ft. than Lake Cochituate at 65 ft., but to the surprise of the author, it was found to be about 10° warmer (see Table No. 5). As the ice breaks up later in the season and the period of circulation takes place in warmer weather and is perhaps more prolonged on account of the mass of water to be affected, it is probable that by the time stagnation of the lower layers begins, the bottom has reached 54 .

The winter temperatures taken under the ice are very interesting, showing that the mass of water cools down to a lower point than does Lake Cochituate. This the author believes is accomplished by the winds and currents before the surface freezes over. The bottom temperatures, 36.65° at 100 ft. and 36.25° at 141 ft., are much below maximum density. At points where there are swift currents and local

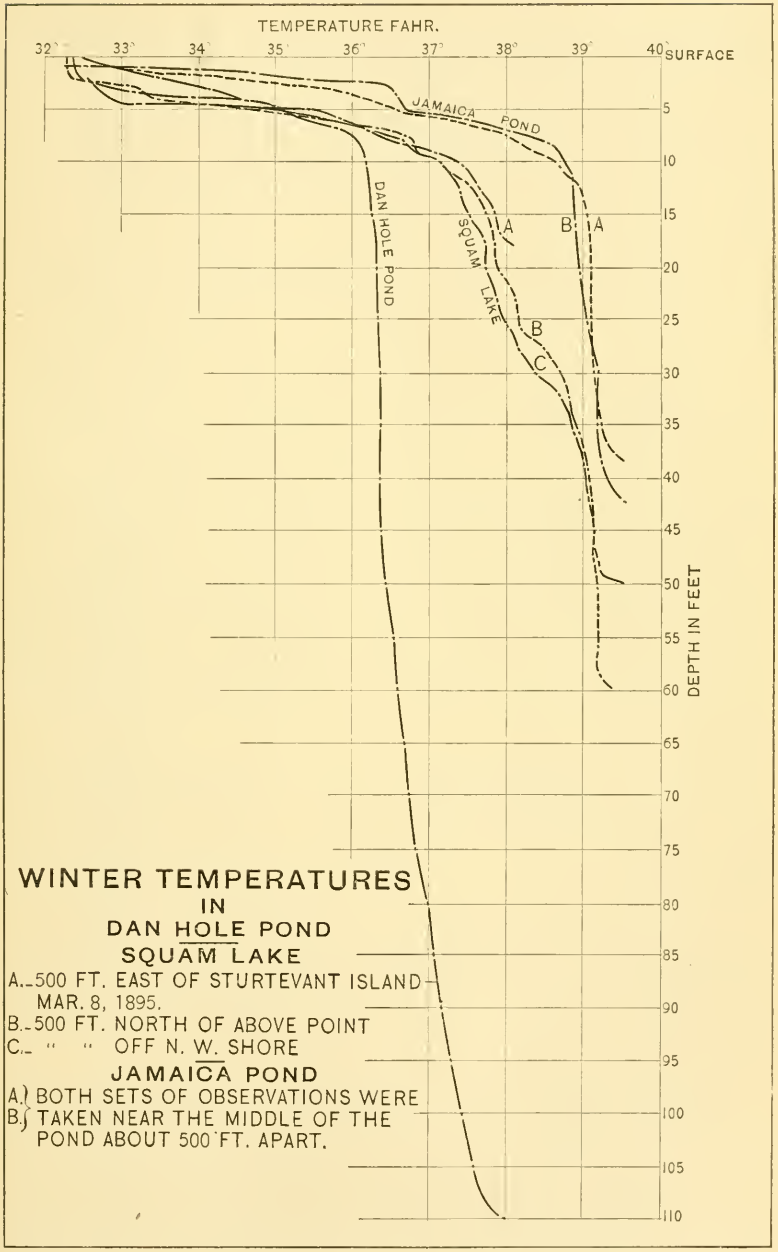

TEMPERATURE FAHR.
32°
33°
34°
35°
36°
37°
38°
39°
40°
SURFACE
5
10
15
20
25
30
35
40
45
50
55
60
65
70
75
80
85
90
95
100
105
110
DEPTH IN FEET
JAMAICA POND
DAN HOLE POND
SQUAM LAKE
A
B
C
A
B
WINTER TEMPERATURES
IN
DAN HOLE POND
SQUAM LAKE
A.-500 FT. EAST OF STURTEVANT ISLAND
MAR. 8, 1895.
B.-500 FT. NORTH OF ABOVE POINT
C.- " " OFF N. W. SHORE
JAMAICA POND
A.) BOTH SETS OF OBSERVATIONS WERE
B.) TAKEN NEAR THE MIDDLE OF THE
POND ABOUT 500 FT. APART.

FIG. 7.

reasons for a different result, the curves change very materially in form, but these are fully shown in the tables and diagrams and require no farther explanation. The inclination of the isotherms in the cross-section of the lake is shown in Fig. 6.

Squam Lake, just north of Lake Winnepesaukee, was found to be at the point of maximum density at the bottom in 59 ft. of water, and at another point, 17 ft. deep, the temperature at the bottom was 37.95 degrees.

Dan Hole Pond, in the same vicinity, and one of the so-called "ponds without a bottom," was found to be 109 ft. deep and to have a temperature of 37.85° at the bottom.

All of the above temperatures, taken under the ice, follow in their curves the same form as in all frozen lakes (see Table No. 6 and Fig. 7).

Lake Superior.—The only observations of this lake that the author has been able to secure were taken in 1871 by the United States Lake Survey, and are given in Table No. 7. The surface and bottom temperatures in August only are given, and show the bottom to be at about 38.8° in 158 fathoms. The surface temperature was from 50 to 53°. The water in this lake is noted for being very cold. As the surface never freezes over it would not be difficult to construct a diagram of the probable ranges in temperature throughout the year, very much as the anatomist constructs a complete skeleton from one or two bones.

Lake Tahoe.—Latitude 39° north, and longitude 120° west. Professor Le Conte, in August, 1873, found the temperature in this lake, at a depth of 1 506 ft., to be 39.2° Fahr.; at 772 ft., 41.0°, and 67.0° at the surface.

F. P. Stearns, M. Am. Soc. C. E., has published in the Report of the Massachusetts State Board of Health, 1890, entitled, "Examination of Water Supplies," an excellent article on the temperature of water. Mr. Stearns' observations on the Upper Mystic Lake, Medford; Jamaica Pond, Boston; and Wenham Lake, Wenham, have been incorporated in Table No. 8.

Professor William R. Nichols read a paper in 1880, before the Boston Society of Natural History, "On the Temperature of Fresh Water Lakes and Ponds," which contains a valuable bibliography on the subject, and is characterized by the deep scientific knowledge possessed by him. Professor Nichols, however, leaned toward the conduction theory.

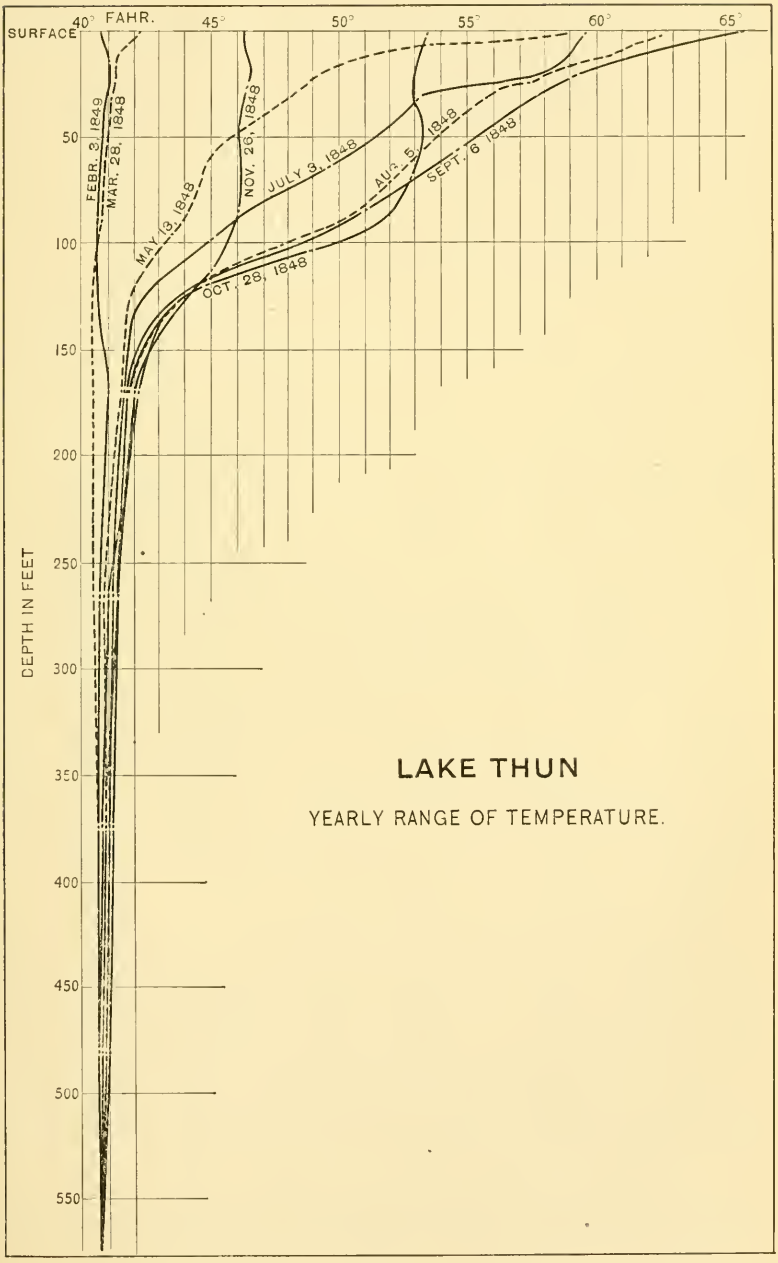

40° FAHR.
45°
50°
55°
60°
65°
SURFACE
50
100
150
200
250
300
350
400
450
500
550
DEPTH IN FEET
FEBR. 3, 1849
MAR. 28, 1848
MAY 13, 1849
NOV. 26, 1848
JULY 3, 1848
AUG. 5, 1848
SEPT. 6 1848
OCT. 28, 1848
LAKE THUN
YEARLY RANGE OF TEMPERATURE.

FIG. 8.

Foreign Lakes.—Extended series of observations on the temperatures of lakes are very rare; in fact, the number of observers who have done valuable work in this direction may be counted on the fingers of one hand. The work of M. H. B. de Saussure, already referred to, is contained in Table No. 9. It will be seen that there are no intermediate temperatures given between the surface and bottom. The results are very close to those obtained by subsequent observers who had the advantage of better instruments, but the observations are now interesting principally from the historical point of view.

One of the most valuable series was taken by MM. C. de Fischer-Ooster and C. Brunner in Lake Thun in 1848–49 (see Table No. 10 and Fig. 8). It was probably the first to give an idea of the seasonal changes going on in a large body of water, and, although taken like M. de Saussure's, without the advantage of deep-sea thermometers, it bears every stamp of accuracy. A number of protected thermometers were fastened to a line and allowed to remain all night, when they were drawn up rapidly and read. The series consists of nine soundings taken in different months of the year and extending to a depth of 550 Swiss feet. They show a variation at the bottom during this time of only about 0.2° Fahr., the mean being 40.7°, slightly above the point of maximum density.

The authors refrained from speculation in their admirable paper,* but they pointed out clearly by means of diagrams the diminution of temperature throughout the mass in the winter and the heating of the lower strata long after the surface had begun to cool off. It is shown that the maximum temperature at depths of 10 to 80 ft. occurs in September; at 80 to 120 ft. in October, and at 120 to 350 ft. in November, while below this point the changes are very slight.

The following extract is from the paper:

"In glancing over this table it is surprising to note the slowness with which the action of the summer sun penetrates into the interior of the water. Although the object of this memoir is to give only such facts as were observed, I cannot but draw attention to these results, which afford information concerning the conduction of heat by water. In closing, I must note that the level of uniform temperatures commences very nearly at a depth of 150 m., and that this temperature varies between 4.8° and 4.9° Cent."

---

* Published in the *Mémoires* of the Societé de Physique et d'Histoire Naturel, Geneva; Vol. XII.

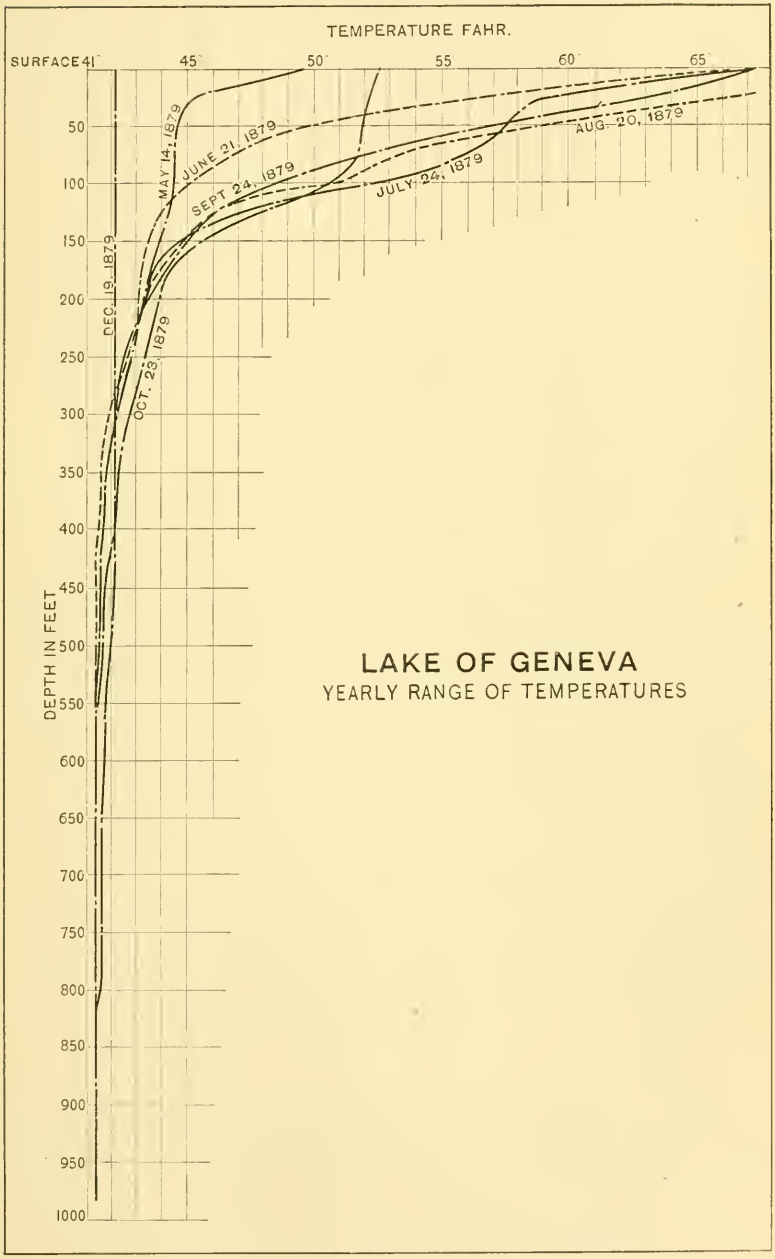

TEMPERATURE FAHR.
SURFACE 41°
45°
50°
55
60°
65°
50
100
150
200
250
300
350
400
450
500
550
600
650
700
750
800
850
900
950
1000
DEPTH IN FEET
MAY 14, 1879
JUNE 21, 1879
SEPT 24, 1879
JULY 24, 1879
AUG. 20, 1879
DEC. 19, 1879
OCT. 23, 1879
LAKE OF GENEVA
YEARLY RANGE OF TEMPERATURES

FIG. 9

The above facts are apparently a strong argument in favor of the theory of conduction, but if the heating of the lower layers in Lake Cochituate occurs from the mingling which goes on between the layers as they approach each other in temperature, as the author has endeavored to show, there is every reason for supposing that the same action goes on in Lake Thun, and that conduction has nothing to do with the question.

Lake of Geneva.—M. F. A. Forel has been one of the most voluminous writers on the temperature of lakes. His observations embodied in Table No. 11 and Fig. 9 have been confined principally to the Lake of Geneva, which has a depth of about 1 000 ft. While not as complete as the Fischer-Ooster observations, they are extremely interesting, confirming in many respects the work of the former and showing the similarity of the results in the two lakes.

He has shown,* from some soundings taken in 1885, that the isotherms of the Lake of Geneva are not horizontal, but inclined, being considerably warmer at a depth of 60 m. at the Chillon end of the "Grand-Lac" than at the same depth at the entrance to the "Petit-Lac." He ascribes this action to the sediment brought into the lake by the Rhone, which he says gives a greater density to the water than that due to its temperature alone, and so it sinks deeper and warms the deep layers of the lake at one end.

In another article † Forel has given the following results of several years of observations:

	Centigrade.	Fahrenheit.
1879	5.2°	41.4°
1880	4.6°	40.3°
1881	4.8°	40.6°
1883	5.0°	41.0°
1884	5.4°	41.7°
1885	5.6°	42.1°
1886	5.3°	41.5°

He admits that the cooling of a lake in the winter is easily explained on the theory of thermic conduction, but that the heating process, which goes down to the most profound depths, is not so easily

* See *Comptes Rendus*, Vol. CII, 1886, p. 712.
† See *Comptes Rendus*, Vol. CIII, p. 47.

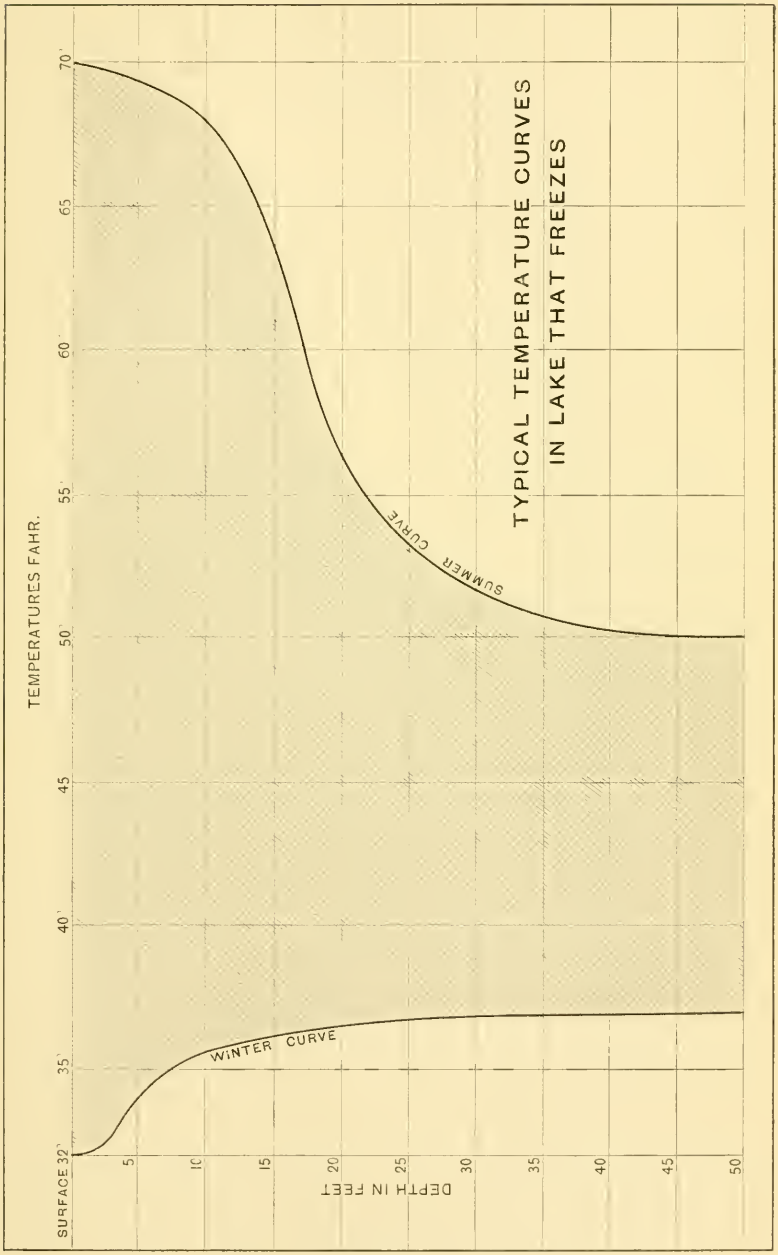

TYPICAL TEMPERATURE CURVES
IN LAKE THAT FREEZES
TEMPERATURES FAHR.
SURFACE 32°
35°
40°
45°
50°
55°
60°
65°
70°
SUMMER CURVE
WINTER CURVE
5
10
15
20
25
30
35
40
45
50
DEPTH IN FEET

FIG. 10.

accounted for by the same theory. He cites a notable instance in his experience when, after a very severe period of high winds and gales in April, he found the temperature had penetrated to extraordinary depths, while the surface had grown cooler. He concludes that the heating is due to a mechanical mixture under the action of the winds.

Table No. 12 contains some interesting observations cited by M. Forel.* Three of the observations were taken in the winter, under the ice, and their close agreement with the results of the author, obtained on the Boston Water-Works and in Lake Winnepesaukee, can be seen at a glance. M. Forel shows that in the Lake of Geneva the temperature at the bottom during the severe winter of 1879-80 did not fall below 5° Cent., and that it is doubtful whether the water in the open lake has ever reached 4° Cent. (39.2° Fahr.).

Conclusions.—It remains now, in view of all the foregoing facts, brought together from different parts of the world, to discuss some of the points connected with the temperature of lakes not already made clear; of course, the discussion can apply only to lakes situated in a climate somewhat like that of Boston. In deep lakes covered with ice in the winter, there are two lines or curves of profile which confine the variations of temperature within certain limits—the winter curve and the summer curve; and these curves nearly meet at the bottom, if it is a very deep lake, and are separated by a considerable interval as the lakes become more shallow. A common shape for the winter curve is to start with 32° at the surface, and increase rapidly to 36° at a depth of 5 ft., and then very gradually towards 39.2° at the bottom. The summer curve may begin at the surface with 75°, and then, by a reverse curve, a type easily recognized from the diagrams, reduce to 45-50° at the bottom (see Fig. 10). The variations from this profile are pronounced, but easily explained by situation, extent of surface and depth. A very remarkable variation from the winter curve is seen in the several sets of observations taken in Lake Cochituate and other lakes, under the ice, which give bottom temperatures higher than 40° (see Fig. 11). As the temperature under the ice is necessarily near 32°, it is evident that the point of maximum density in these cases is some distance above the bottom. Crystal Lake, shown in Fig. 11 and embodied in Table No. 6, is a small body of clear water fed

* See *Comptes Rendus*, Vol. XC, 1880, p. 322.

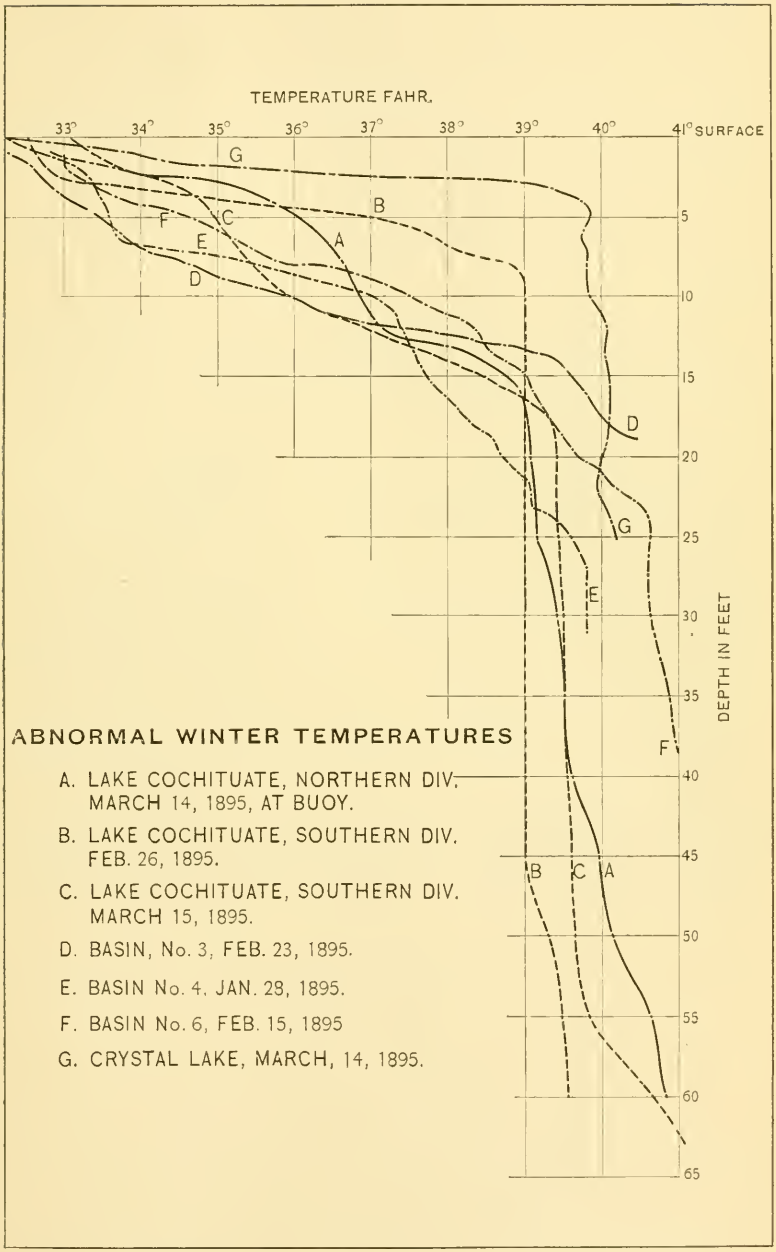

TEMPERATURE FAHR.
33°
34°
35°
36°
37°
38°
39°
40°
41° SURFACE
5
10
15
20
25
30
35
40
45
50
55
60
65
DEPTH IN FEET
ABNORMAL WINTER TEMPERATURES
A. LAKE COCHITUATE, NORTHERN DIV. MARCH 14, 1895, AT BUOY.
B. LAKE COCHITUATE, SOUTHERN DIV. FEB. 26, 1895.
C. LAKE COCHITUATE, SOUTHERN DIV. MARCH 15, 1895.
D. BASIN, No. 3, FEB. 23, 1895.
E. BASIN No. 4. JAN. 28, 1895.
F. BASIN No. 6, FEB. 15, 1895
G. CRYSTAL LAKE, MARCH, 14, 1895.

FIG. 11.

entirely by springs. In this case it is possible that the ground-water entering the pond gradually raised the temperature beyond the point of maximum density, and that this heating effect continued upward nearly to the ice line. This is the most exaggerated case that the author has found. The corroborative evidence of all these curves is beyond question. A study of the tables shows that this condition of unstable equilibrium may continue for a long time, but it is certainly very curious, and, as far as the author knows, has never been noticed before.

In a lake of the first order, like that of Geneva, the winds produce a mechanical mixture of the layers to a considerable depth below the surface, even in the summer, say to 150 ft. in depth, as is evident from the tables. The return currents, being much warmer and lighter than the lower layers, must pass over them. The smaller the lake, the less these mechanical effects are felt. In Lake Cochituate, for instance, they are hardly found 20 ft. below the surface; but that this extensive heating to considerable depths in large lakes is not due to conduction seems to be proved by the fact that at 65 ft. depth conduction has no effect in seven months' time in altering the bottom temperatures of Lake Cochituate. When the water is of nearly the same temperature throughout, the return currents may pass to the bottom and circulation may occur to great depths. This uniformity of temperature in ordinary lakes takes place at two points in the yearly profile, and at these times the wind effects are felt to great depths. In a very large lake which does not freeze, this period occurs during the winter only, but these changes are probably prolonged, and, at any rate, are sufficient to prevent periods of stagnation extending from one year to another.

LIST OF TABLES.

PAGE

1. Weekly Mean Temperatures (Fahr.) for Five Years, 1890–94, Boston Water-Works. 92
2. Temperatures (Fahr.) Lake Cochituate, 1891, 1892, 1893 and 1894..... 94
3. Lake Cochituate. Temperatures (Fahr.) taken with the Thermophone. 98
4. Boston Water-Works. Temperatures (Fahr.) taken with the Thermophone..... 100
5. Lake Winnepesaukee. Temperatures (Fahr.) taken with the Thermophone. 102
6. Dan Hole Pond, Squam Lake, Jamaica Pond and Crystal Lake Temperatures (Fahr.) taken with the Thermophone..... 104
7. Temperatures in Lake Superior, U. S. Lake Survey.—Nichols..... 106
8. Temperatures of Some Lakes in Massachusetts.—Stearns. 106
9. Temperatures of Swiss Lakes.—H. B. de Saussure 107
10. Temperatures of Lake Thun, 1848, 1849.—Fischer-Ooster and Brunner..... 107
11. Temperatures of Lake Geneva.—F. A. Forel..... 108
12. Temperatures of Some Swiss Lakes.—F. A. Forel..... 109

TABLE No. 1.—Weekly Mean Temperatures (Fahr.) for Five Years, 1890–94, Boston Water-Works.

Date.	Lake Cochituate.			Basin 2.		Basin 3.		Basin 4.		Chestnut Hill Reservoir.			Brookline Reservoir.	Boston Tap Water.	
	Surface.	Mid-depth.	Bottom	Surface.	Bottom.	Surface.	Bottom.	Surface.	Bottom.	Surface.	Bottom.	Gate-House.	Gate-House.	City.* "Common."	Maltapan.†
Jan. 1....	35.2	36.2	37.7	35.4	36.5	35.0	37.3	35.0	37.1	36.8	37.9	36.8	37.0	40.7	42.1
8....	35.8	36.9	38.4	33.3	35.4	33.7	35.3	34.1	36.3	35.6	36.4	35 5	36.7	39.0	41.2
15....	34.6	35.6	37.5	33.2	34.7	33.3	36.0	34.1	36.9	35.2	35.5	35.7	36.6	39.3	40.7
22....	35.5	36.5	38.0	33.7	35 4	33.6	36.1	34.6	36.9	34.0	36.5	35.7	36.4	38.2	39.3
29....	35.6	36.5	37.6	33.7	35.2	33.9	36.7	34.5	37.3	34.4	35.7	36.6	36.5	38.4	39.1
Feb. 5....	34.4	36.4	37.8	33.9	36.1	33.8	37.1	35.0	37.2	35.3	35.9	36.2	36.6	37.4	38.5
12....	34.6	37 0	38.1	33.4	36.6	33.9	37.0	34.0	37.6	34.1	36 1	36.1	36.4	38 1	38.5
19....	35.6	37.5	38.3	34.0	35.6	33.3	37.2	34.1	38.2	33.2	35.7	35.7	36.4	37.3	37.5
26....	34 9	36 8	38.1	33.4	35.7	33.4	35.9	33.8	37.3	35.3	35.9	35.1	36.2	37.5	38.0
Mar. 5....	34.7	37.4	38.5	33.8	36.0	33.5	37.1	34 4	37.9	32.9	35 0	35.9	36.1	38.1	38.0
12	36.7	38.9	37.4	35.0	36.4	34.3	36.6	35.3	38.4	38.5	38.5	37.0	36.8	38.8	38.5
19....	37.0	38.2	38.6	37.0	38.1	36.5	38.3	38.1	39.7	40 2	40.6	37.7	38.1	39.5	38.8
26....	38.2	38.7	39.0	39.0	39.8	37.7	39.3	40.4	40.3	40.6	40.9	39.2	39.6	40.2	38.5
Apr. 2....	40.0	40.1	40.1	43.4	43.2	41.4	42.6	41.6	40.8	42.4	42.5	42 6	42.5	41.5	41.3
9....	42.2	41.2	41.7	45 9	45.3	44.9	44.9	44.1	43.6	44.6	44.8	45.5	43.9	44.5	41.4
16....	46.3	43.6	43.3	49.4	48.9	48.6	47.6	47.0	44.1	47.7	46.4	47.0	46.4	46.1	42.8
23....	49 0	44.9	43.8	52.4	51.9	50.7	50.5	50.8	47.5	50.7	48.8	50.0	49.8	50.4	45.0
30....	51.8	45.9	44.0	56.3	55.3	54.9	54.1	52.9	48.7	54.2	51.6	53.1	53.0	51.2	46.0
May 7....	55.9	46.5	43.8	57.8	57.0	56.7	54.7	56 0	50.0	56.7	54.2	55.7	54.4	54.3	48.6
14....	58.2	47.5	44.1	61.6	59 7	60.1	57.7	59.4	50.3	61.2	55.1	58.0	57.7	56.4	50.0
21....	61.6	47.8	44.0	62.3	60.1	60.7	58.6	60.9	50.4	59.9	57.2	60.0	58.5	56.9	50.1
28	64.8	48.0	44.2	65.7	62.2	63.9	60.4	65.8	53.1	64 3	58.1	61.7	60.8	61.0	52.9
June 5...	67.7	48.4	44.2	68.1	63.1	68.6	61.4	68.3	52.4	66.9	59.0	64.5	64.0	62.8	53.7
12	71.6	50.4	44.2	73.7	64.7	72.6	63.0	70.5	52.4	70.0	59.1	65.1	66.0	66.2	56.9

19....	72.4	49.7	44.2	73.0	67.5	71.8	64.2	73.0	52.3	72.4	60.0	67.5	67.6	68.1	58.5
26....	71.2	49.1	44.3	72.5	68.0	70.9	62.7	71.2	52.3	71.2	58.2	68.6	69.6	68.5	59.8
July 3....	71.8	49.4	44.1	75.0	69.8	73.7	65.5	73.6	52.1	74.6	58.5	70.8	70.0	69.7	60.2
10....	75.1	49.3	43.9	76.3	71.0	75.1	66.2	75.9	52.9	75.7	59.3	71.5	71.7	71.1	61.1
17....	74.7	49.8	44.3	76.0	72.4	75.2	66 3	75.5	53.1	75.3	57.6	72 2	74.2	73.1	63.2
24....	74.9	49.7	44.4	75.8	71.7	75.0	66.9	76.0	52.9	75.3	60.4	73.1	74.1	73.6	64.0
31...	75.9	48.9	44.0	77.7	74.5	76.7	68.2	76.4	52.6	76.2	61.2	74.8	73.5	72.7	63.9
Aug. 7....	76 9	49.9	44.1	75.9	73.3	75.4	67.6	75.0	53.1	75.5	62 2	74.4	73.6	73.1	64.7
14....	74.5	49.8	44.0	73.6	72.3	73.2	67.2	73.9	52.5	74.8	63.1	73.8	74.4	73.2	64.7
21....	73.7	49.9	44.0	73.1	71.8	72.6	67.5	73.1	52.8	74.1	61.7	72.9	72.7	72.4	65.0
28....	69.8	49.6	44.5	71.3	70 1	70.8	67.9	70.8	52.5	69.9	62.3	71.7	70.7	70.4	64.2
Sept. 4....	68.9	49.3	44.3	68.6	67.4	68.7	67.1	68.1	54.1	69.7	63.6	69.4	69.1	68.7	63.1
11...	68.8	49.0	44.1	68.7	67.7	69.4	67.6	68.8	55.3	69.9	64.6	68 4	68.4	68.9	62.9
18....	68.3	49.3	44.1	68.2	67.2	67.8	66.5	66.8	57.1	68.6	63.9	67.9	67.5	67.1	62.2
25....	66.5	49.2	44.1	66.4	65.6	66.6	65 2	65.5	58 4	66.2	63.9	67.1	66.8	65.8	61.9
Oct. 2...	62.5	49.2	44.4	60.7	60.3	61.7	61.0	60.4	55.6	63.4	61.8	62.5	62.9	62.9	60.6
9...	58.9	49.4	44.2	57.4	57.2	58.1	58.1	56.8	55.5	59.8	59 5	59.8	59.4	60.2	59.5
16....	56.0	51.4	44.1	53.9	53.6	53.8	53.8	54.3	53.6	56.5	56.5	55.9	56.4	56.8	57.6
23....	52.6	51.7	44.4	49.6	49.7	50.5	50.5	51.7	51.9	53.5	53.3	53.4	52.9	54.3	56.3
30....	50.5	49.7	44.3	47.6	47.7	47.4	47.5	49.0	49.1	51.4	52.0	50.7	50.6	52.2	55.0
Nov. 6....	48.1	47.9	45.5	44.1	44.0	45 6	45.4	47.2	47.2	49.0	49.1	48.9	48.0	48.7	52.4
13....	46.7	46.3	45.6	40.7	40.7	42.5	42.4	44.7	44.2	45.4	45.7	45.8	45 0	47.8	51.0
20....	45.0	43.9	45.2	40.4	40.9	40.6	40.9	42.0	43.6	43.0	43.3	44.6	44.2	45.7	49.7
27....	43.4	43.0	43.7	36.0	36.3	37.2	37.4	40.2	39.7	40.6	40.9	39.9	40.6	43.4	48.1
Dec. 4....	38.4	38.8	41.1	35.0	35 6	35.8	36.5	38.4	38.4	38.0	37.9	38.4	38.8	41.3	45.8
11...	37.0	37.2	41.0	35.4	35.9	35.3	36.4	36.9	38.6	38.5	38.7	36.7	37.3	42.0	44.1
18....	37.1	37.1	39.4	35.7	36.1	34.6	36.8	37.9	39.3	35.9	37.1	36.6	36.4	39.9	43.4
25....	35.9	36.7	39.1	34.7	36.4	35.3	37.1	36.0	38.3	36.9	37.3	36.2	37.1	39.2	42.4

* Five miles from gate-house. † Seven miles from gate-house.

TABLE No. 2.

Temperatures (Fahr.). Lake Cochituate. 1891.

Depth in Feet....	0	5	10	15	20	25	30	35	40	45	50	55	Bot.
April 14th.....	45.2	45.0	44.6	44.6	44.5	44.0	44.0	43.8	43.8	43.8	43.8	43.8	43.8
" 21st.....	55.3	54.5	53.5	48.8	47.3	45.4	44.4	44.1	43.7	43.5	43.2	43.4	43.9
" 29th.....	53.0	53.0	53.0	53.0	53.0	52.0	49.0	46.0	45.0	44.7	44.5	44.0	44.0
May 5th.....	53.2	53.2	53.2	53.2	52.6	52.1	46.8	45.0	44.8	44.0	43.8	43.6	43 4
" 12th.....	58.1	55.5	55.2	52.0	51.2	48.9	45.8	45.0	44.8	43.8	43.7	43.4	43.7
" 19th.....	55.5	55.4	55.2	55.0	54.1	50.8	48.1	45.8	45.0	44.3	43.6	43.5	43.4
" 26th.....	63.0	63.0	61.0	58.0	54.2	53.4	48.0	46.7	46.7	45.8	45.0	44.5	44.0
June 2d.....	67.0	65.6	62.7	56.9	54.3	50.7	48.3	46.9	46.0	46.0	44.6	44.5	44.3
" 9th.....	67.0	66.9	66.0	59.3	53.9	50.2	47.6	46.7	45.5	44.9	44.8	44.6	44.4
" 18th.....	71.4	71.1	60.6	57.5	53.9	50.3	48.8	47.7	47.0	46.8	45.0	44.2	43.9
" 23d.....	70.5	69.6	67.0	60.0	55.0	52.0	48.9	47.7	46.6	46.0	45.3	45.0	44.5
" 30th.....	69.7	68.5	67.6	62.5	56.9	52.7	50.9	49.7	48.5	47.6	46.4	45.5	44.8
July 6th.....	70.3	70.2	69.7	64.1	53.6	50.5	48.0	47 6	46.1	45.2	45.0	44.7	43.8
" 13th.....	73.8	72.7	70.0	63.9	54.3	50.9	48.8	47.7	47.4	46.0	45.0	44.2	*43.1
" 21st.....	77.0	76.4	74.8	63.5	53.8	51.1	50.0	48.6	47.8	45.4	44.6	44.0	43.2
" 28th..... ...	74.5	74.4	73.0	64.2	55.6	51.1	48.9	47.8	45.6	44.9	44.0	43.5	43.3
Aug. 4th.....	72.1	72 0	71.4	64.2	57.7	50.9	48.9	47.8	46.7	45.3	44.6	43.8	43.2
" 11th.....	76.8	76.1	74.0	62.4	56.5	52.4	51.3	48.8	47.4	46 3	44.8	43.4	43.4
" 18th.....	74.5	74.4	73.0	64.4	56.2	51.1	49.3	48.2	46.9	45.8	44.3	43.8	43.1
" 25th.....	77.6	77.0	75.2	67.8	58.2	53.7	51.3	48.6	47.0	45.7	44.2	43.1	43.1
Sept. 10th.....	72.7	71.3	68.2	67.9	59.4	51.4	50.6	48.2	47.6	47.4	46.8	45.4	45.4
" 17th.....	69.5	69.0	68.5	67.5	59.0	51.5	50.0	48.5	47.4	45.7	44.6	44.2	44.2
" 23d.....	70.9	70.5	70.0	68.0	61.5	52.0	50.5	48.9	47.6	46.2	45.5	44.0	44.0
" 29th.....	71.8	71.6	71.0	65.5	55.8	50.9	49.5	47.6	45.9	44.7	44.1	43.5	43.5
Oct. 6th.....	67 0	66.9	66.8	66.1	58.4	50.5	48.3	47.5	46.8	45.7	45.1	44.6	44.6
" 13th.....	57.5	57.5	57.5	57.5	54.1	50.8	48.0	46.2	45.4	44.2	43.6	43.4	43.4
" 20th.....	55.6	55.6	55.6	55.4	55.4	54.6	49.4	46.4	45.6	44.7	43.9	43.4	43.4
" 27th.....	49.6	49.6	49.6	49.6	49.6	49.4	49.4	48.3	45.7	44.9	44.1	43.4	43.4
Nov. 3d.....	46.5	46.5	46.5	46.5	46.5	46.5	46.5	46.5	45.3	45.1	44.0	43.6	43.6
" 10th.....	45.0	45.0	45.0	45.0	45.0	45.0	45.0	45.0	45.0	45.0	45.0	45.0	45.0
Average	64.4	63.9	62.6	58.8	54.1	50.6	48.5	47.1	46.2	45.3	44.6	44.0	43.8

* Represents observation taken with particular care.

TABLE No. 2—(*Continued*).

TEMPERATURES (FAHR.). LAKE COCHITUATE 1892.

DEPTH IN FEET....	0	5	10	15	20	25	30	35	40	45	50	55	Bot.
April 9th.....	45.4	45.4	45 4	45.3	45.2	44.8	44.6	44.4	44.4	44 3	44.1	44.0	44.0
" 26th.....	49.6	49.5	49.2	48.7	47.5	46.4	45.6	45.5	45 3	45.1	45.0	44.8	44.5
May 3d.....	50.9	50.5	50.2	50.0	49.6	48.5	46.0	45.5	45.3	45.2	45.0	44.9	44.8
" 10th.....	54.2	54 1	54.0	53.5	52.3	50.9	47.7	46.9	46.4	45.9	45.2	44 8	†44.3
" 17th.....	56.9	56.9	56.8	56.6	56.3	53.0	50.4	48.9	47.2	46.4	45 7	45.1	44.6
" 24th.....	55.3	55.3	55.2	55.2	55.0	54.5	50.5	48.6	47.3	46.1	45.6	44.8	44.3
June 1st.....	67.2	65.5	62.2	59.5	57.5	55.5	50.8	50.0	49.5	48.6	46.5	45.3	44.1
" 7th.....	70.7	70.5	69.1	60.5	56.0	54.4	51.5	50.0	49 8	48.5	45.9	44.8	44.1
" 14th	76.0	73.2	71.4	62.0	57.3	55.2	51.0	48.3	47.8	46.9	46.4	45.2	44.1
" 21st.....	75.6	75.4	74.5	64.9	59.2	55.1	53.0	51.5	49.8	48.9	47.7	45 5	44.0
July 6th.....	72.5	72.1	71.6	69.2	58.0	54.3	51.2	49.8	48.6	47.4	46.2	45.7	44.0
" 13th.....	73.3	72.8	72.2	70.8	59.1	54.5	51.7	49.4	47.8	46.9	46.1	45 2	44.0
" 19th.....	76.4	75.0	72.3	70.2	60.5	54.5	52.0	50.7	49.1	47.8	46.6	45.3	44.1
" 25th.....	78.7	76 3	74.6	71.0	58.0	53.7	50 5	49.8	48.3	47.6	46.8	45.3	*44.1
Aug. 2d.....	74.0	73.5	72.6	70.5	59.6	53.0	50.5	49.3	48.6	47.9	46.5	45.8	44.0
" 9th.....	76.0	75.8	75.5	70.6	57.7	53.6	51.0	50.2	49.6	48.3	47.5	45.8	44.0
" 16th.....	77.3	77.3	76.0	73.0	59.0	55.0	51.0	49.0	47.5	47.0	46.0	45.8	44.1
" 22d	76.7	76.3	76.1	71.5	60.6	55.5	50.6	49.5	48.6	47.9	47.3	46.7	44.7
" 30th.....	68.5	68.3	68.2	68.1	61.0	53.5	51.0	49.7	47.8	46.9	46.3	45.2	44.3
Sept. 7th.....	68.3	68.2	68.1	68.0	60.2	54.2	50.5	49.0	47.8	46.7	45.9	45.1	44.3
" 13th.....	67.5	66.8	65.5	64.3	63.6	51.7	50.5	49.4	48.6	47.1	45.9	45.2	43.9
" 20th.....	67.5	67.2	66.8	66.4	63.6	53.7	50.5	48.4	47.1	46.3	45.6	44.8	44.1
" 27th.....	64.6	64.5	64.4	64.4	64.4	60.0	57.2	49.6	47.9	46.5	45.8	45.0	44.3
Oct. 4th.....	59.9	59.7	59.6	59.5	59.4	56.7	51.1	49.4	47.5	46.6	45.9	45.2	44.3
" 11th.....	56.8	56.8	56.7	56.6	56.5	56.1	51.3	49.4	47.5	46.7	45.9	45.2	44.3
" 18th.....	56.5	56.5	56.5	56.5	56.3	55.8	52.4	49.2	47 4	47.5	45.8	45 1	*44.3
" 25th.....	53.0	52.8	52.8	52.7	52.6	52.5	52.4	49.9	47.5	46.7	45.8	45.1	44.4
Nov. 1st.....	50.6	50.6	50.6	50.5	50.4	50.2	50.1	49.5	47.9	46.6	45.8	45.1	44.5
" 8th.....	48.7	48.7	48.7	48.6	48.5	48.4	48.2	47.5	46.4	45.7	45.3	44.8	44.3
" 15th.....	46.4	46.4	46.4	46.4	46 4	46.4	46.4	46.4	46.4	46.4	46.4	46.4	46.4
Average.....	63.8	63.4	62.7	60.8	56.4	53.0	50.4	48.8	47.7	46.8	46.0	45.2	44.3

* Represents observations taken with particular care. † Bottom, May 5th, 44.5.

TABLE No. 2—(*Continued*).

TEMPERATURES (FAHR.). LAKE COCHITUATE. 1893.

DEPTH IN FEET....	0	5	10	15	20	25	30	35	40	45	50	55	60
April 18th.....	42.0	42.0	42.0	42.0	42.0	42.0	42.0	42.0	42.1	42 1	42.2	42.2	42.3
" 25th.....	44.4	44.2	44.0	43 8	43.6	43.4	43.3	43.2	43.1	43.0	42 9	42.9	42.8
May 2d.....	46.7	46.4	46.0	45.6	45.2	44.9	44.7	44.5	44.3	44 1	44.0	43.9	43.8
" 9th.....	50 0	48.9	48.4	47.7	47.3	46.5	46.2	45.9	45.8	45.5	45.3	45.8	44.4
" 16th.....	59.3	58.4	53.5	48.8	48.3	48.0	47.3	47.0	47.0	47.0	47.0	46.8	45 5
" 23d.....	64.2	62.8	58.0	54.2	48.5	48.0	47.8	47.2	47.0	47.0	47.0	47.0	45.5
" 31st.....	63.1	62.0	61.3	60.0	51.7	48 0	47.8	47.5	47.5	47.5	47.5	47.2	45.3
June 6th.....	71.3	70.0	64.3	55.2	49.3	48.1	47.0	46.8	46.5	46.3	45.8	45.4	45.1
" 13th.....	73.3	73.2	72.0	59.7	52.2	49.5	49.0	48.7	48.3	48 3	48.3	48.2	45.2
" 20th.....	72.8	72.6	71.5	64.2	63.5	58.0	48.7	48.0	47.5	47.0	47.0	47.0	45.3
" 26th.....	67.8	67.6	67.0	58.4	52.7	49.2	47.6	47.1	46.8	46.3	45.9	45.5	45.1
" 30th.....	63.1	62.5	61 6	57.8	51.1	49.6	47.3	46.9	46.5	46.2	45.7	45.4	44.9
July 11th.....	75.2	74.5	73.8	67.4	51.5	49.8	48.0	47 6	47.2	46.5	45.9	45.4	44.9
" 18th.....	76 3	75.1	74.8	64.2	52.5	50.1	48.4	47.9	47.5	46.7	46.2	45.6	*45.0
" 25th.....	74.5	73.8	73.0	64.2	51.5	49.3	48.1	47.8	47.2	46.7	46.1	45.6	45.0
Aug. 1st	73.5	72.9	72.3	64.8	52.4	49.3	48.0	47.7	47.1	46.5	45.8	45.4	44.9
" 8th.....	74.4	74.0	72 0	66.0	54.0	53.0	52.2	47.8	47.0	46.5	45.9	45.4	45.1
" 15th.....	74.6	74.1	71.5	69.0	56.0	53.2	48.9	47.8	47.1	46.5	45.9	45.4	44.9
" 26th.....	71.3	70.2	70.0	68.5	56.4	52.8	51.2	48.8	47.3	47.0	47.0	45.7	44.8
" 29th.....	69.8	67.5	62 3	58.4	52.5	49.7	48.9	48.6	47.9	47.3	46.5	45.8	44.9
Sept. 6th.....	68.2	68.1	67.9	66.8	62.5	56.2	49.1	48.3	47.7	46.9	46.4	45.8	45.0
" 12th.....	66.8	66.4	66.4	65.0	58 3	52.5	50.3	48.3	48.0	47.2	47.0	47.0	44.8
" 19th.....	65.2	65.2	65.2	64.5	58.3	53.8	51.2	49.8	48.5	48.5	48.5	47.8	44.8
" 26th.....	63.2	63.1	63.0	62.6	62.1	55.8	49.5	48 4	47.5	46.8	46.1	45 6	44.9
Oct. 3d.....	58.4	58.2	58.0	57.8	57.5	53.7	49.4	48.2	47.5	46.9	46.3	45.6	44 9
" 10th.....	59.0	59.0	58.8	58.2	58.2	54.5	51.5	50.8	50.8	50.8	50.8	49.8	45.1
" 17th	56.5	56.5	56.5	56.5	56.4	54.3	52.8	49.0	47.9	47.1	46.2	45 7	44.9
" 24th.....	56.4	56.3	56.1	56.0	55.8	55 1	54.4	49.7	48.5	47.6	46.9	46.2	45.4
" 31st.....	53.4	53.4	53.4	53.4	53.4	53.4	53.4	50.1	48.5	47.7	46.9	46.2	45.0
Nov. 7th.....	50.6	50.6	50.6	50.6	50.6	50.6	50.2	50.0	48.5	47.0	47.0	47.0	45.7
" 16th.....	46.1	46.1	46.1	46.1	46.1	46.1	46.1	45.9	45.8	45.7	45.7	45.5	45.3
" 21st.....	45 0	45.0	45.0	45.0	45.0	45.0	45.0	45.0	45.0	45.0	45.0	45.0	45.2
Average.....	62.4	61.9	60.8	57 6	52.7	50.5	48.7	47.5	47.0	46.5	46.3	45.9	45.0

* Represents observation taken with particular care.

TABLE No. 2—(*Continued*).

TEMPERATURES (FAHR.). LAKE COCHITUATE. 1894.

DEPTH IN FEET	0	5	10	15	20	25	30	35	40	45	50	55	Bot.
April 17th.....	43.8	43.5	43.5	42.8	42.5	42.1	42.1	41.7	41.7	41.7	41.7	41.7	41.7
" 24th.....	50.2	50.0	49.2	48.8	47.3	45.8	44.5	43.0	42.2	41.8	41.8	41.8	*41.3
May 1st.....	57.0	56.5	54.9	50.0	47.1	45.7	44.0	43.6	43.2	42.9	42.5	42.1	41.5
" 8th.....	61.1	60.7	60.0	54.4	48.6	46.8	44.3	43.9	43.3	42.8	42.4	42.1	41.5
" 16th.....	61.2	61.0	59.7	58 8	56.9	47 2	45.9	45 0	44.6	43.9	43.7	43.7	41.8
" 23d.....	61.5	60.3	59.1	57.6	51.0	47.2	44 6	43.9	43.3	42.8	42.3	41.9	41.5
" 31st.....	62.3	61.9	61.5	57.8	51.5	47.2	45.5	44.6	43.9	43.4	42.7	42.3	41.8
June 6th.....	62.5	62.1	61.4	58.6	52.3	48.0	45.6	44.8	44.2	43.8	42.9	42.5	41.9
" 15th.....	71.5	69.5	67.6	59 8	53.4	49.4	47.0	46.1	45.2	44.4	43.2	42.7	42.2
" 21st.....	75.7	74.2	71.6	60.5	54.2	50 5	48.1	47.2	45.9	44.7	43.5	42.9	42.4
" 26th.....	73.3	73.0	71.2	62.7	53.5	51.7	48.2	47.5	47.3	47.1	46.8	46.8	42.3
July 5th.....	77.1	75.8	72.6	65.0	52.5	49.1	47.3	46.0	45.2	44.5	43.9	43.1	42.6
" 11th.....	76.6	76.2	74.5	70.6	56.8	52.3	48.9	48.7	48.5	48.5	48.1	47.5	42.7
" 18th.....	77.6	77.1	76.0	69.5	56.0	50.7	48.1	47.8	47.5	47.2	46.6	46.0	42.8
" 25th.....	78.6	78.1	77.4	68.5	55.2	49.2	47.5	46.9	46.4	45.8	45.0	44.5	42.9
Aug. 1st.....	79.5	79.0	78.8	67.2	54.3	47.6	46.9	46.1	45.3	44.5	43.6	43.2	42.8
" 7th.....	78.5	77.3	75.8	64.7	52.5	49.2	47.5	46.4	45.6	44.7	44.1	43.5	43.0
" 15th.....	73.6	72.8	72.3	65.4	50.8	49.2	47.8	46.5	45.6	44.9	44.3	43.7	43.1
" 22d.....	70.2	70.1	70.0	69.2	54.9	50.5	47.4	46.5	45.8	45.0	44 2	43.6	43.0
" 29th.....	70.5	70.2	70.1	70.0	51.5	49.0	47.1	46.2	45.4	44.5	43.8	43.3	42.9
Sept. 5th.....	68.8	68.6	67.9	67.5	52.2	49.7	47.4	46.3	45.1	44.4	43·5	43.2	42.9
" 12th.....	69.7	68.9	67.7	65.6	53 1	49.9	47.3	46.1	45.2	44.4	43.7	43.3	42.8
" 19th.....	68.3	67.6	66.8	66.4	52.2	49.7	45.5	45.0	44.4	43.9	43.5	43.2	42.9
" 26th.....	65.4	64.9	63.4	62.3	52.5	49.3	46 4	45.5	44.8	44.1	43.2	43.2	42.9
Oct. 3d.....	61.8	61.7	61.6	61 5	53.5	50.5	46.8	46.1	45.3	44.4	43.6	43.2	43.0
" 10th.....	59 2	59.1	58.9	57.8	49.0	48.2	46.9	46.0	45.3	44 6	43.9	43.4	43.1
" 17th.....	53.5	53.5	53.5	53.3	53.5	52.9	52.4	48.1	46.5	45.2	44.4	43.7	43.3
" 24th.....	52.7	52.7	52.6	52.5	52.4	50.0	48.6	47.7	46.3	45.1	44.3	43.8	43.4
" 31st.....	51.5	51.5	51.4	50.8	50.3	49.5	48.2	47.7	46.6	45.4	44.5	43.8	43.3
Nov. 7th.....	47.4	47.4	47.4	47.4	47.4	47.4	47.3	47.3	47.3	47.3	47.3	47.3	47.3
Average.....	66.4	64.8	63.9	60.2	52.0	48.8	46.8	45.9	45.2	44.6	44.0	43.6	42.7

* Bottom, 41.4.

TABLE No. 3.

Lake Cochituate—Temperatures (Fahr.) taken with the Thermophone.

	Northern Division.														Middle Div.	Southern Division.		
	December 21, 1894.						Feb. 26, 1895.	March 14, 1895.		Mar. 29, 1895.	Apr. 10, 1895, 11 A. M.	Apr. 13, 1895, 11 A. M.	Apr. 15, 1895.	Apr. 18, 1895.	Apr. 22, 1895.	Feb. 26, 1895.	February 26, 1895.		Mar. 15, 1895.
Depth in feet.	At buoy near gate-house.	100 ft. west of buoy.	Half way from buoy to opposite shore.	300 ft. from shore opposite the gate-house.	Point near the shore opposite the gate-house.	Point near the shore opposite the gate-house.	At buoy.	At buoy.	Half way from buoy to opposite shore.	At buoy.	At buoy.	At buoy.	At buoy.	At buoy.	At buoy.		About the middle of the Southern Division.	Near the B. & A. R. R. bridge.	About the middle of the Southern Division.
Surface..	32.8	33.50	33.80	33.50	33.80	33.40	32.45	33.05	32.15	.....	33.15	39.70	39.85	42.85	44.55	32.25	32.55	32.15	32.15
1.....	35.2	35.40	35.50	36.60	36.70	36.40	32.55	.....	.....	33.20	.....	.....	.....	.....	.....	32.65	32.65	32.25	32.85
2.....	35.7	36.60	37.60	37.55	37 70	37.40	32.75	33.80	32.85	35.60	38.40	39.70	39.85	42.35	44.45	32.75	32.80	32.55	33.80
3.....	37.3	37.40	38.05	37.65	37.85	37.90	33 40	....	.....	36.50	.....	.....	.....	.....	.....	33.70	33.60	33.60	
4.....	37.7	37.45	38.20	37.70	38.00	38.15	34.55	35 70	35.40	36.80	38.60	39 70	39.75	42.15	44.45	34.55	35.10	34.95	34.80
5.....	37.8	37.55	38.30	37.70	38.05	38.35	35.15	.....	.....	37.00	.....	.....	.....	.....	.....	35.55	36.90	35.70	
6.....	37.8	37.75	.....	.....	.....	.....	35.55	36.25	36.15	37.00	38.70	39.70	39.70	41.85	44.45	35.95	37.70	36.90	35.10
7.....	37.9	37.85	.....	.....	.....	.....	36.75	.....	.....	37.00	.....	.....	.....	.....	.....	36.05	38.10	37.50	
8.....	37.9	37.90	.....	.....	.....	.....	38.20	36.65	36.60	37.00	38 70	39.30	39 70	41.75	43.55	36.30	38.80	37.90	35.45
9.....	37.9	37.90	.....	.....	.....	.....	38.95	.....	.....	36.90	.....	.....	.....	.....	.....	36.50	39.00	38.30	
10.....	38.0	37.85	38.40	37.75	38.20	38.40	39.00	36.85	36.80	36.90	38 70	39.15	39.75	41 65	43.55	36.55	39.00	38.50	35.95
11.....	.....	.....	.....	.....	.....	.....	39.00	.....	.....	37.00	.....	.....	.....	.....	.....	36.55	.....	38.60	
12.....	.....	.....	.....	.....	.....	.....	39.05	37.15	37 10	37.20	.....	39.10	39.65	41.60	43.55	36.70	.....	38.70	36.95
13.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	37 30	.....	.....	.....	.....	.....	36.90	.....	38.80	
14.....	.....	.....	.....	.....	.....	.....	.....	38.45	38.25	37.30	.....	.....	39.55	41.35	43.55	37.05	.....	38.85	37.95
15.....	38.1	37.85	38.40	37.75	38.25	bot. 38.45	39.10	38.75	.....	37.30	38.70	39.05	.....	.....	.....	37.40	39.00	39.10	
16.....	.....	.....	.....	.....	.....	.....	.....	38.85	38.95	37.50	.....	.....	39.50	41.15	43.55	37.55	.....	39.10	38.85
17.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	37.60	.....	.....	.....	.....	.....	37.80	.....	bot. 39.30	

18.....	.....	.....	.....	.....	.....	.....	.....	39.00	39.05	37.80	.....	.....	39.50	41.05	43.35	37.95	.....	.....	39.35
19.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	37.80	.....	.....	.....	.....	.....	38.10			
20.....	38.2	37.85	38.40	37.85	38.25	.....	39.30	39.05	39.05	37 90	38.60	39 05	39.35	41.05	43.00	38.20	39.00	.....	39.40
21.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	38.20	.....	.....	.....	.....	.....	38.30	.....	.....	
22.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	38.50	.....	.....	.....	.....	.....	38.40	.....	.....	39.40
23.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	38.70	.....	.....	.....	.....	.....	38.45			
24.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	38.80	.....	.....	.....	.....	.....	38.45			
25.....	38.25	37.90	38.45	37.85	.....	.....	39.30	39.15	39.20	38.80	38.60	39.05	39.35	41.00	42.45	38.50	39.00	.....	39.45
26.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	38.50			
27.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	38.60			
					bot.														
28.....	.....	.....	.....	.....	38.30	.....	.....	.....	.....	39.30	.....	.....	.....	.....	.....	38.60			
29.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	38.65			
30.....	38.35	37.95	38.50	37.85	.....	.....	39.40	39.45	39.45	39.30	38.60	39.00	39.35	40.65	41.55	38.65	39.00	.....	39.50
31.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	38 70			
32.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	38.70			
33	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	38.80			
																bot.			
34.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	38.85			
35.....	38.40	38.00	38.50	37.90	.....	.....	39.40	39.55	39.60	39.30	38.90	39.00	39.35	40.50	41.35	.....	39.00	.....	39.55
40.....	38.55	38.20	38.55	38.00	.....	.....	39.50	39.60	39.65	39.50	38.95	39.00	39.35	40.35	41.25	.....	39.00	.....	39.55
45.....	38.70	38.40	38.70	38.70	.....	.....	39.80	39.95	39.90	39.60	39.00	38.75	39.35	40.30	41.15	.....	39.00	.....	39.60
				bot.															
46½.....	.....	.....	.....	38.90															
48.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	38.65							
50.....	38.90	38.45	39.25	.....	.....	.....	40.20	40.15	40.30	39.90	39.20	38.65	39.35	40.30	41.05	.....	39.30	.....	39.65
51.....	39.10	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....		
	bot.																		
52.....	39.15	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	38.65							
			bot.																
53.....	.....	.....	39.30																
		bot.							bot.										
54.....	.....	38.90	.....	.....	.....	.....	.....	.....	40.55										
									mud.										
55.....	.....	.....	.....	.....	.....	.....	40.60	40.65	40.65	40.20	39.40	38.65	39.35	40.25	41.05	.....	39.50	.....	39.85
57.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	39.70								
							bot.	bot.											
58.....	.....	.....	.....	.....	.....	.....	40.80	40.75	.....	40.40	39 80	38.65							
								mud.		bot.							bot.		
60.....	.....	.....	.....	.....	.....	.....	.....	40.85	.....	40.70	40.00	.....	39.35	40.20	41.05	.....	39.55	.....	40.65
										mud.									
61.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	40.75	40.00								
											bot.	bot.	bot.	bot.	bot.				
62.	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	40.60	38.55	39.35	40.15	41.05				
																			bot.
63.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	40.95
														mud.	mud.				mud.
														40.05	40.55	.....	.....	.....	41.05

TABLE No. 4.

BOSTON WATER-WORKS.—TEMPERATURES (FAHR.) TAKEN WITH THE THERMOPHONE.

	Basin 2.					Basin 3.					Basin 4.			Basin 6.			
	Jan. 25, 1895.	February 25, 1895.			April 11, 1895, 4 P. M.	Jan. 25, 1895.	February 23, 1895.			April 12, 1895. 10 A. M	Jan. 28, 1895.	Feb. 12, 1895.	April 11, 1895. 12 M.	February 15, 1895.			April 11, 1895. 2 P. M.
Depth in feet.	Buoy about 150 feet from dam	About 100 feet from gate-house.	Buoy about 150 feet from dam.	Two-thirds of distance from dam towards bridge.	At buoy.	Buoy 500 feet from dam.	Buoy 500 feet from dam.	Half way from dam to railroad bridge.	Near railroad bridge.	At buoy.	Buoy 150 feet from gate-house.	Buoy 150 feet from gate-house.	At buoy.	Near south end of dam.	Buoy near north end of dam.	Half way from dam to island.	At buoy.
Surface..	32.50	.....	.....	.....	43.15	33.40	.....	.....	.....	39.45	32.30	32.00	38.75	.....	.....	.....	38.20
1.....	32.90	32.55	32.55	32.25	.....	33.50	32.35	32.25	.....	.....	32.70	32.20	.....	33.10	33.00	32.75	
2.....	33.10	32.65	32.75	32.35	43.15	33.60	32.50	32.65	32.65	39.45	33.20	32.20	38.60	34.10	33.00	32.85	38.15
3.....	33.30	32.80	32.90	32.65	.....	33.70	33.50	32.80	32.80	.....	33.40	32.20	.....	34 45	33.40	32 90	
4.....	33.40	32.90	33.00	32.80	43.10	33.90	33.50	33.10	33.05	39.45	33.50	32.20	38.55	34.75	33.85	33.00	38.15
5.....	33.40	33.10	33.10	33.00	.....	34.00	33.60	33.45	33.70	.....	33.60	32.60	.....	35.15	34.60	33.60	
6.....	33.40	33.25	33.20	33.20	42.80	34.80	34.00	33.75	34.10	39.45	33.70	32.90	38 40	35.45	35.05	34.15	38.10
7.....	33.45	33.60	33.35	33.60	.....	35.20	34.10	34.00	34.55	.....	34.30	33.60	.....	36.45	35.45	34.65	
7½.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	35.35	
8.....	33.70	33.80	33.60	33.85	42.75	36.30	34.20	34.65	35.05	39.25	35.50	33.80	38.40	36.85	35.95	35.75	37.90
8½.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	36.75		
9.....	34.50	34.80 bot.	34.55	35.05	.....	36.90	35.25	35.10	35.55	.....	36.20	34.30	.....	37.30	37.05	36.15	
10.....	35.30	34 95	36.95	36.85 bot. 37.00	42.65	37.40	35.70	35.95	36.05	39.25	37.00	35.10	38.35	37.90	37.50	36.60	37.90
10½.....	.....	.....	.....	.....													
11.....	36.80	.....	38.00	.....	.....	37.80	36.70	36.35	36.85	.....	37.30	35.90	.....	38.40	37.90	36.95	
11.....	38.50																
12.....	39.15	.....	.....	.....	42.60	38.50	37.40	37.40	38.20	39.25	37.30	36.50	38 30	38.80	38.35	37.35	37.95

			bot.														
12½			38.50				38.70	38.60	38.60		37.50	36.70		39.20	38.50	37.50	
13	39.30																
	bot.																
14	39.30				42.45		39.50	39.40	39.00	39.20	37.60	36.95	38.30	39.50	38.75	37.90	38.00
15						39.20	39.70	39.60	39.60		37.70	37.20		39.90	39.00	38.20	
					bot.												
16					42.45		39.80	39.80		39.15	37.90	37.40	38.30	40.15	39.10	38.40	38.00
									bot.								
16½									39.60								
17							39.90	39.90			38.10	37.70		40.40	39.20	38.65	
						bot.								bot.			
18						39.70	40.00	40.05		39.10	38.30	37.85	38.30	40.60	39.35	38.75	38.00
								bot.									
19							40.00	40.50			38.60	38.20			39.50	38.80	
20							40.10			39.10	38.70	38.40	38.30		39.70	38.90	38.00
							bot.										
21							40.25				38.90	38.70			40.00	38.95	
22										39.10	39.10	38.90	38.30		40.20	39.10	38.00
23											39.10	39.20			40.50	39.50	
										bot.							
24										39.10	39.40	39.35	38.30		40.60	39.75	38.00
																bot.	
25											39.60	39.40			40.60	39.80	
26											39.70	39.50	38.30		40.60		38.00
27												39.55			40.60		
28											39.80	39.60	38.30		40.60		38.00
29												39.70			40.60		
												bot.					
30											39.80	39.70	38.30		40.65		38.00
											bot.						
31											39.80						38.00
32													38.30				
33															40.80		38.00
34													38.30				
35															40.90		38.00
36													38.30				
37															40.95		38.00
38													38.30				
															bot.		
38½															41.00		37.95
40													38.30				37.90
42													38.35				
													bot.				
44													38.40				37.95
46																	38.05
48																	38.05
																	bot.
50																	38.05

TABLE No. 5.—Temperatures (Fahr.). Lake Winnepesaukee, Thermophone.

Between Sandy and Welch Islands. Aug. 25, 1892.		North and N. E. of Rattlesnake Island.	Aug. 27, 1892.	North of Barndoor Island.	March 4, 1895.	Between Baker's Island and Ship Island.	March 5, 1895.	Between Rattlesnake and Parker islands.	March 5, 1895.	Near Rattlesnake Island.	March 5, 1895.	"The Broads."	March 7, 1895.	Between Welch and Sandy islands.	March 7, 1895.
Surface.	73.0	Surface.	68.5	2 ft.	32.95	1	32.20	5	32.85	5	32.85	0	32.10	0	32.25
40 ft.	64.5[1]	25 ft.	68.3	3	33.10	5	32.75	7	33.10	7½	33.10	2	32.35	2	32.35
80 "	56.0[2]	50 "	59.0	4	33.20	7	33.10	10	33.20	10	33.25	4	32.90	4	32.90
		65 "	57.1	5	33.20	10	33.30	12	33.20	12½	33.30	6	33.00	6	33.00
		75 "	56.2	6	33.80	12	33.60	15	33.20	15	33.30	8	33.05	8	33.15
		105 "	55.0	7	33.95	14	33.70	17	33.30	20	33.35	9	33.05	10	33.15
		112 "[3]	55.0	8	34.00	16	33.80	20	33.40	25	33.75	10	32.95	12	33.20
		125 "[3]	55 0	9	34.00	18	33.95	25	33.90	30	34.05	11	33.00	14	33.25
		135 "[3]	55.0	10	34.00	20	34.05	30	34.10	35	34.30	12	33.00	16	33.35
		145 "[3]	54.0	11	34.00	22	34.10	35	34.40	40	34.55	14	33.00	18	33.35
		.....	.....	12	34.00	24	34.10	40	34.55	45	34.70	16	33.00	20	33.40
		.....	.....	13	34.00	29	34.45	45	34.90	50	34.85	18	33.10	22	33.45
		.....	.....	14	34.00	32	34.55	50 bot.	35.00	55	35.00	20	33.25	24	33.50
		.....	.....	15	34.00	35	34.65	.....	.....	60	35.10	22	33.35	26	33.55
		.....	.....	16	34.05	40	34.80	.....	.....	65	35.25	24	33.50	28	33.65
		.....	.....	17	34.05	45	35.00	.....	.....	70	35.30	26	33.60	30	33.80
		.....	.....	18	34.05	50	35.10	.....	.....	75	35.30	28	33.80	32	33.90
		.....	.....	19	34.15	55	35.10	.....	.....	80	35.35	30	33.90	34	33.95
		.....	.....	20	34.15	60	35.30	.....	.....	85	35.40	32	34.05	36	34.00
		.....	.....	22	34.25	65	35.40	.....	.....	90	35.45	34	34.15	38	34.20
		.....	.....	25	34.35	70 bot.[4]	35.40	.....	.....	95	35.55	35 bot	34.25	40	34.30
		.....	.....	27	34.55	.....	.....	.....	.....	100	35.65	mud.	34.30	42	34.35
		.....	.....	30	34.55	.....	.....	.....	.....	105	35.75	.....	.....	44	34.40
		.....	.....	32	34.65	.....	.....	.....	.....	110	35.80	.....	.....	46	34.50
		.....	.....	34	34.70	.....	.....	.....	.....	115	35.90	.....	.....	48	34.65
		.....	.....	36	34.70	.....	.....	.....	.....	120	35.95	.....	.....	50	34.65
		.....	.....	38	35.00	.....	.....	.....	.....	125	35.95	.....	.....	52	34.75
		.....	.....	40	35.00	.....	.....	.....	.....	130	36.05	.....	.....	54	34.85
		.....	.....	42	35.10	.....	.....	.....	.....	135	36.15	.....	.....	56	34.85
		.....	.....	44	35.10	.....	.....	.....	.....	140	36.25	.....	.....	58	34.90
		.....	.....	46	35.10	.....	.....	.....	.....	141 bot.	36.25	.....	.....	60	35.00
		.....	.....	48	35.20	.....	.....	.....	.....	142 mud[5]	36.70	.....	.....	62	35.05
		.....	.....	50	35 20	.....	.....	.....	.....	.....	.....	.....	.....	64	35.10
		.....	.....	52 bot.	35.25	.....	.....	.....	.....	.....	.....	.....	.....	66	35.10
		.....	.....	mud.	35.60	.....	.....	.....	.....	.....	.....	.....	.....	68	35.25
		.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	70	35.25
		.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	72	35.25
		.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	74	35.35
		.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	76	35.40
		.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	79 bot.	35.50
		.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	mud.[6]	35.70

TABLE No. 5.—Temperatures (Fahr.). Lake Winnepesaukee, Thermophone—(*Continued*).

East of Long Island.	March 7, 1895.	200 ft. nearer Ringged Island.	March 7, 1895	Between Bear and Five-Mile islands.	March 7, 1895.	Between Beaver and Black Cat islands.	March 7, 1895.	Near wharf at Center Harbor.	March 8, 1895.	Between Weir's and Pitchwood Island.	March 8, 1895.	North West Bay entrance.	March 8, 1895.	Near Meredith village.	March 8, 1895.
0	32.20	0	32.10	0	32.30	0	32.20	0	32.45	0	32.20	0	32.35	0	32.20
2	32.20	2	32.25	2	32.45	2	32.25	2	32.45	2	32.25	2	32.40	2	32.45
4	32.50	4	32.60	4	32.90	4	32.65	3	32.70	4	32.60	4	32.70	4	33.20
6	32 80	6	32.90	6	33.20	6	33.00	4	32.80	6	33.40	6	33.10	6	33.35
8	33.05	8	33.10	8	33.30	8	33.20	5	33.10	8	33.55	8	33.40	8	33.40
10	33.10	10	33.15	10	33.40	10	33.35	6	33.25	10	33.95	10	33.75	10	33.45
12	33.20	12	33.20	12	33.45	12	33.55	7	33.35	12	34.20	12	33.90	12	33 55
14	33.30	14	33.30	14	33.50	14	33.60	8	33.45	14	34.30	14	34.20	14	33.85
16	33.35	16	33.35	16	33.55	16	33.75	9	33.55	16	34.55	16	34.40	16	33.90
18	33.55	17	33.40	18	33.60	18	33.80	10	33.55	18	34.65	18	34.50	18	34.25
18.5	34.75	17.5	33.50	20	33.70	20	33.85	11	33.60	20	34.70	20	34.65	20	34.35
19	35.00	18	33.60	25	33.85	25	33.90	12	33.60	22	34.80	22	34.75	22	34.55
19.5	35.25	18.5	33.90	30	34.00	30	34.15	14	33.70	24	34.80	24	34.80	24	34.65
19.7	35.75	18.7	34.65	35	34.20	35	34.30	16	33.85	26	34.85	26	34.85	26	34.70
20	36.25	19	35.40	40	34.50	40	34.55	17	34.05	28	34.90	28	34.90	28 bot.	34.75
20.5	36.70	19 5	36.00	45	34.80	45	34.95	18	34.15	30	34.95	30	34.95	mud.	35.25
21	36.85	20	36 35	50	35.25	47.5	35.60	19	34.40	32	35.00	32	35.05	.....	.....
21.5	36.85	20.5	36.50	55	35.55	50	35.75	20	34.95	34	35.05	34	35.10	.....	.....
22	36.90	21	36.75	60	35.70	52.5	36.10	21	35.90	36	35.10	36	35.10	.....	.....
23	36.95	22	37.00	65	35.75	55	36.20	22	37.30	38	35.20	38	35.15	.....	.....
24	37.15	24	37.05	70	35.85	57.5	36.30	23	37.80	40	35.25	40	35.20	.....	.....
26	37.20	26	37.15	75	36.10	60 bot.	36.35	24	38.60	42	35.30	42	35.25	.....	.....
28	37.20	28	37.20	80	36.15	mud.	36.60	25	39.05	44	35.35	44	35.25	.....	.....
30	37.25	30	37.20	85	36.25	.....	.....	26	39.25	46	35.35	46	35.30	.....	.....
32	37.25	32	37.25	90	36.35	.....	.....	26.5 bot.	39.30	47 bot.	35.35	47 bot	35.35	.....	.....
33 bot.	37.25	34	37.25	95	36.50	.....	.....	mud.	39.35	mud.	35 55	mud.	35.35	.....	.....
mud.	37.25	36	37.25	100 bot.	36.65	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
.....	.....	40	37.30	mud.	36 95	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
.....	.....	42	37.35	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
.....	.....	44 bot.	37.50	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
.....	.....	mud.	37.80	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....

[1] Mean of two observations. [2] Mean of four observations. [3] Bottom temperatures. [5] Thermometer at bottom, 35.70? [6] Thermometer at bottom, 35.5?

TABLE No. 6.

Temperatures (Fahr.) Taken With the Thermophone.

Dan Hole Pond.		Squam Lake.						Jamaica Pond, Boston, Mass.			Crystal Lake, Newton Centre, Mass.	
¼ mile from east end. Feet.	March 6, 1895.	500 ft. east of Sturtevant Island.	March 8, 1895.	North of last station.	March 8, 1895.	Northwest shore.	March 8, 1895.		*a.* 500 ft	*b.* apart.	Feet.	March 14, 1895.
0	32.50	2.....	32.55	0.....	32.25	0.....	32.40	0.....	32.4	.....	0.....	32.15
2.....	33.45	4.	32.85	2.....	32.30	2.....	32.45	1.....	.....	32.2	1.....	33.90
3.....	34.10	5..	35.45	3.....	33.15	3.....	32.95	1½.....	.....	34.7	1½.....	34.30
4.....	34.50	6.....	35.85	4.....	33.45	4.....	34.65	2.....	34.2	35.1	2.....	35.75
5.....	35.20	7.....	36.15	5.	34.65	5.....	35.15	2½.....	.....	36.1	2½	37.10
6.....	35.35	8.....	36.45	6.....	35.70	6.....	35.70	3.....	35.7	36.5	3.....	39.15
7.....	35.80	9.....	37.10	7.....	36.30	7.....	36.20	4.....	36.2	36.6	3½.....	39.50
8.....	36.05	10.....	37.35	8.....	36.80	8.....	36.70	5.....	36.6	36.7	4.....	39.75
9.....	36.10	11.....	37.55	9.....	36.85	9.....	36.85	6.....	37.35	37 40	5.....	39.85
10.....	36.15	12.....	37.60	10.....	37.15	10.....	37 10	7.....	37.75	37.95	6.....	39.75
12.....	36.20	13.....	37.70	11.....	37.25	11.....	37.25	8.....	38.15	38.45	6½.....	39.75
15.....	36.25	14.....	37.80	12.....	37.50	12.....	37.35	9.....	38.35	38.79	7.....	39.80
17.....	36.30	15.....	37.85	14.....	37.70	14.....	37.45	10.....	38.65	38.75	8.....	39.80
20.....	36.30	16..... ..	37.90	16.....	37.80	16.....	37.60	11.....	38.75	38.80	10.....	39.85
25.....	36.30	17 bot.....	37.95	18.....	37.85	18.....	37.75	12.....	38.95	38.85	12.....	40.05

30.....	36.35	Mud.....	38.10	20.....	37.90	20.....	37.75	15.....	39.05	38.85	14.....	40.05
40.....	36.35	.....	.....	22.....	38.05	22.....	37.85	17.....	39.10	.. ..	16.....	40.10
50.....	36.45	.....	.....	24.....	38.15	24.....	37.95	20.....	39.10	38.95	18.....	40.10
55.....	36.55	.....	.....	26.....	38.20	26.....	38.10	25.....	39.10	39.05	20.	40.00
60.....	36.60	.....	.....	28.....	38.55	28.....	38.20	30.....	39.15	39.15	22.....	39.95
65.	36.70	.....	.....	30.....	38.75	30.....	38.40	35.....	39.25	39.20	24.....	40.15
70.....	36.75	.....	.....	32.....	38.85	32.....	38.70	37.....	39.35 bot.	.....	Mud.....	40.20
75.....	36.85	.....	.....	34.....	38.85	34.....	38.85	40.....	.. ..	39.30		
80.....	37.00	.....	.....	36.....	39.00	36.....	38.90	41.....	.....	39.45 bot.		
85	37.05	.....	.....	38.....	39.05	38.....	39.00	Mud	39.55	39.60		
90.....	37.15	.....	.....	40.....	39.10	40.....	39.05	.....	.....	.....		
95..... .	37.25	.....	.....	42.....	39.10	42.....	39.10	.....	.....	.....		
100.....	37.45	.....	.....	44.....	39.15	44.....	39.15	....	.....	.....		
105.....	37.60	.....	.....	46.....	39.15	46.....	39.15	.....	.....	.....		
109 bot.....	37.85	.....	.....	48.....	39.15	48.....	39.20	.....	.....	.....		
Mud.	38.05	.....	.....	50.....	39.20	49 bot.....	39.25	.....	.....	.....		
.....	.....	.....	.....	52.....	39.20	Mud	39.60	.....	.....	.....		
.....	.....	.....	.....	54.....	39.20	.....	.....	.....	.....	.....		
.....	.....	.....	.....	56.....	39.20	.....	.....	.....	.....	.....		
.....	.....	.....	.....	58.....	39.20	.....	.....	.....	.....	.....		
.....	.....	.....	.....	59 bot.....	39.25	.....	.....	.....	.....	.....		
.....	.....	....	.....	Mud	39.40	.....	.....	.....	.....	.....		

Small pond fed by springs.

TABLE No 7.

Temperatures in Lake Superior. U. S. Lake Survey, 1871.—Nichols.

Date.	Latitude.	Longitude.	Depth in fathoms.	Depth in feet.	Temperature at surface in degrees.		Temperature at bottom, in degrees.	
					Fahr.	Cent.	Fahr.	Cent.
Aug. 7...	46° 53′	86° 43′	127	762	56.5	13.6	38.8	3.8
.....	to	to	129	774	56.0	13.3	38,8	3.8
.....	47° 23′	86° 15′	121	726	53.0	11.7	39.1	3.9
.....	.....	.....	131	786	50.1	10.1	38.8	3.8
.....	.....	.....	74	444	50.8	10.4	38.8	3.8
Aug. 11...	48° 18′	86° 15′	59	354	55.8	13.2	41.3	5.2
Aug. 15...	48° 20′	87° 3′	105	630	49.7	9.8	39.0	3.9
.....	to	to	121	726	49.5	9.7	39.2	4.0
.....	47° 45′	87° 6′	168	1008	49.2	9.5	39.0	3.9
.....	.....	.....	116	696	50.2	10.1	39.3	4.1
.....	.....	.....	136	816	52.2	11.2	38.9	3.8
.....	.....	.....	158	948	52.9	11.6	33.4 ?	0.8
Aug. 18...	48° 9′	88° 15′	47	282	51.8	11.0	36.8	2.7
.....	to	to	129	774	52.7	11.5	38.9	3.8
.....	47° 36′	87° 42′	136	816	52.3	11.3	38.8	3.8
.....	.....	.....	134	804	50.1	10.1	38.9	3.8
.....	.....	.....	82	492	51.5	10.8	39.0	3.9
Aug. 22...	Near Copper Harbor		148	888	.....	.....	38.7	3.7
.....	.....	.....	20	120	51.3	10.7	39.7	4.3
.....	.....	.....	61	366	55.3	12 9	39.2	4.0
Aug. 29...	48° 42′	87° 47′	73	438	55.0	12.8	38.9	3.8

TABLE No. 8.

Temperatures of Some Lakes in Massachusetts.—F. P. Stearns.

	Upper Mystic Lake, Medford.	Jamaica Pond, Boston.	Wenham Lake, Wenham.
Date.	August 23d, 1889.	August 21st, 1889.	July 24th, 1889.
Area.	200 acres.	69 acres.	320 acres.
Maximum depth.	87 ft.	57 ft.	47 ft.
Average depth.	.....	27 ft.	
1 ft.....	73 4°	74.1°	74.7°
10 "	71.6	70.7	74.4
15 "	68.9	64.8	74.3
20 "	63.3	52.0	
23 "	.....	47.5	
25 "	56.3	46.0	59.0
30 "	49.6	43.7	
35 "	46.4	.....	53.6
40 "	45.5	.....	
45 "	44.4	42.4	49.6
60 "	42.8		
82 "	42.8		

TABLE No. 9.

Temperatures of Swiss Lakes.—H. B. de Saussure.

Lake.	Date.	Depth. Feet.*	Temperature.	
			Surface. Fahr.	Bottom. Fahr.
Geneva	February 6th, 1777	1 013	42.1	41.7
Geneva	August 5th, 1779	160	70.2	43.0
Thun	July 7th, 1783	373	66.2	41.0
Brientz	July 8th, 1783	533	66.9	40.6
Lucerne	July 28th, ——	640	68.5	40.8
Constance	July 25th, 1784	394	64.6	39.7
Maggiore	July 19th, 1783	357	77.0	44.1
Neuchatel	July 17th, 1779	346	73.6	41.2
Bienne	July 20th, 1779	231	69.3	43.4
Annecy	May 14th, 1780	174	57.9	42.1
Bourget	October 6th, 1784	256	64.2	42.1

*De Saussure's depths are given in Swiss feet which have here been multiplied by 0.9382929, to reduce to English measures.

TABLE No. 10.

Lake Thun.—Fischer-Ooster and Brunner, 1848-49.

English feet.	March 28, 1848.	May 13.	July 3.	August 5.	Sept. 6.	Oct. 28.	Nov. 26.	Feb. 3, 1849.
.....	42.3	59.1	59.6	62.8	65.6	53.4	46.3	40.8
10.7	41.4	51.4	59.0	60.4	61.8	53.2	46.3	41.0
21.3	41.3	49.2	57.2	57.3	59.1	53.0	46.4	41.2
32.0	41.2	48.0	53.2	55.5	57.7	53.0	46.2	41.0
42.6	41.1	46 6	52.1	54.4	56.2	53.2	46.1	40.7
63.9	40.9	44.8	49.6	52.6	53.8	53.0	46.1	40.7
85.3	40.8	44.2	46.3	50.7	50.9	52.1	46.1	40.7
127.9	40.4	41.8	42.3	43.7	43.4	43.6	44.0	40.7
170.5	40.4	41.5	41.4	41.8	41.7	42.1	42.1	41.0
266.5	40.4	40.8	41.0	41.1	41.4	41.0	41.3	40.7
373.0	40.6	40.9	40.9	41 1	41.1	40.8	40.7	40.6
479.6	40.7	40.7	40.8	40.9	40.8	40.9	40.6	40.7
586.3	40.7	40.7	40.8	40.7	40.8			

Lake Thun is known to have been frozen four times, 1363, 1435, 1685 and 1695.

TABLE No. 11.

Temperatures of Lake Geneva.—F. A. Forel.

Depth.		1879.								1880.	
Meters.	Feet.	May 14th.	June 21st.	July 24th.	August 20th.	September 24th.	October 23d.	December 10th.	December 24th.	January 15th.	January 30th.
0	0	49.65	66.40	67.30	71.60	66.55	52.50	42.10	41.70	41.35	41.00
10	32.8	44.95	54.15	58.30	64.40	61.35	52.00	42.10			
20	65.6	44.60	47.65	56.85	54.85	53.95	51.80				
30	98.4	44.40	45.30	53.05	50.90	48.75	50.72				
40	131.2	44.25	43.90	46.20	45.70	45.70	47.10				
50	164.0	43.70	43.35	44.05	44.40	44.60	44.80				
60	196.8	.....	.....	43.35	43.15	43·50	43.90	42.10			
70	229.6	43.00	43.00	42.80	42.80	42.80					
80	262.4	42.60	42.45	42.45	42.45	42.25	43.15				
90	295.2	42.25	41 90	42.10	42.10	41.90	42.80				
100	328.0	41.90	41.70	41.90	41.90	41.70	42.45				
110	360.8	41.70	41.55	41.70	.....	.....	42.25	42.10			
120	393 6	41.55	41.55	41.55	41.70	41.55	42.10				
130	426.4	.....	41.35	41.35	41.55	41.55	41.90				
140	459.2	.....	.....	.....	41.55	41.55	41.70				
150	492.0	.....	.....	.....	.....	41.35	41.70	41.90			
160	524.8	41.35	.....	.....	.....	.....	41.55				
170	557.6	.....	.....	.....	.....	.....	41.35				
200	656.0	.....	.....	.....	.....	41.35	.....	41.55			
220	721.6	41.35									
240	787.2	.....	.....	.....	.....	.....	.....	41.55			
250	820.0	.....	.....	.....	.....	.....	.....	41.35			
260	852.8	.....	41.35								
270	885.6	.....	.....	.....	.....	.....	.....	41.35			
300	984.0	.....	.....	41.35	41.35	.....	41 35				

The surface temperature on January 15th, 1880, was 41.35°, and on January 30th, 41°.

TABLE No. 12.

Temperatures of Some Swiss Lakes.—Forel.

Lake Annecy.			Lake Bourget.		Lake Zurich.		Lake Morat.		
August 13th, 1880.		Sept. 22d, 1883.	September 22d, 1883.		January 25th, 1880.		December 23d, 1879.		Feb 1st, 1880.
Depth. Feet.	Temperature. Fahrenheit.	Temperature. Fahrenheit.	Depth. Feet.	Temperature. Fahrenheit.	Depth. Feet.	Temperature. Fahrenheit.	Depth. Feet.	Temperature. Fahrenheit.	Temperature. Fahrenheit.
Surface.....	65.8	63.5	Surface.....	65.8	Surface.....	36.4	Surface.....	32.7	32.6
32.8.....	64.9	63.5	32.8.....	64.4	32.8.....	36.7	16 4.....	34.9	35.4
65.6.....	50.5	46.4	65.6.....	52.3	65.6.....	37.2	32.8.....	35.6	35.6
98.4.....	44.1	46.4	131.2.....	43.3	98.4.....	37.7	49.2.....	36.0	36.4
131.2.....	42.8	42.8	196.8.....	42.6	131.4.....	38.3	65.6.....	36.4	36.5
164.0.....	41.9	42.8	262.4.....	42.4	164.0.....	38.5	82.0	36.7	36.5
180.4.....	41.9	43.0	377.2.....	42.3	196.8.....	38.7	98.4.....	36.8	36.0
196.8.....	41.0	43.0			229.6.....	38.7	114.8.....	37.0	36.6
			Never frozen.		262.8.....	38.8	131.2.....	37.0	36.9
					295.2.....	38.8	Ice. December 23d, 1879, 4.3 ins.		
					328.0.....	39.0	February 1st, 1880, 14.2 "		
					360.8.....	39.0	Area, 10.6 sq. miles.		
					393.6.....	39.2	Depth, 147 ft.		
					436.2.....	39.2			
					Area, 34 sq. miles.				
					Depth, 468 ft.				
					Frozen once.				

DISCUSSION.

Mr. Fanning. J. T. Fanning, M. Am. Soc. C. E.—This paper emphasizes the necessity of providing outlets at different elevations for reservoirs and particularly storage reservoirs. It is common practice to provide two or three gates. As long ago as 1868 the speaker, realizing this necessity, arranged an adjustable outlet for a storage reservoir of 65 acres area as follows: The main outlet pipe was fitted in a Y joint and placed near the bottom of the reservoir about 20 ft. below the flow line. This adjustable pipe revolved in a vertical plane and was provided with a large strainer bulb and counterweighted so that it could be set readily at any altitude. This was a convenient arrangement by which not only could water be drawn from the level where the best was found, but the strainer could be raised above the water surface and readily cleaned.

Mr. Darrach. C. G. Darrach, M. Am. Soc. C. E.—The paper is no doubt the most exhaustive treatise upon the subject of the temperature of lakes, and the author's theory of semi-annual vertical circulation is new. Even without experimental data, serious thought upon the subject would naturally lead one to suppose that there would be a tendency to such vertical circulation.

In January, 1883, the speaker took some isolated observations upon the temperature of surface and bottom waters in the Schuylkill River, and in three of the Philadelphia reservoirs. The water had been covered with ice for some time previous to the time of observation. The data taken was as follows:

Location.	Air.	Water.			
	Degrees, Fahr.	Surface. Degrees, Fahr.	Bottom. Degrees, Fahr.	Depth. Feet.	Difference.
Fairmount Dam	.....	33.	33.4	12	0.4
Fairmount Reservoir	34.	33.2	33.8	12	0.6
Spring Garden Reservoir	34.	33.4	33.8	16	0.4
Roxborough Reservoir	36.	33.	33.4	18	0.4

It will be noticed that the temperature of the water immediately under the ice was somewhat higher than the observations noted by the author.

In 1881 a difference of 12° Fahr. was noted between the temperature of the surface water and the water at a depth of 50 ft. in Druid Hill Reservoir, Baltimore.

The spring and fall equilibrium in the densities of water in deep reservoirs has a tendency to produce a vertical circulation, but the

speaker was not prepared to admit that there is an entire semi-annual overturning. Mr. Darrach.

In August, 1875, (while in company with Messrs. W. J. McAlpine, J. W. Adams, Milnor Roberts, William E. Morris, Solomon W. Roberts and William H. McFadden, one of the numerous commissions on the water supply of Philadelphia), during a visit to Tumbling Run Dam, a feeder to the Schuylkill Canal, it was noticed that the water drawn from a depth of 50 ft. was very offensive, both in taste and odor. After running a short distance in the brook, the disagreeable taste and odor disappeared. Much discussion ensued among the members of the commission as to the cause of the trouble, the catchment area of the reservoir being at that time free from artificial pollution. The speaker suggested that decaying vegetation, infusoria, etc., sank to the lower waters of the reservoir, where, for lack of heat and air, decomposition was suspended, and the water containing this pollution, after being exposed to the atmosphere, was speedily purified.

In the winter and spring of 1881 the water supply of the city of Baltimore was so offensive that a public outcry resulted, and venders of pure spring water reaped a harvest. The water of the city storage reservoirs and mains was drawn off and the trouble temporarily ended.

At the instance of the late Robert K. Martin, M. Am. Soc. C. E., the chief engineer, the speaker visited Baltimore in 1882 and suggested to him that he should waste all surplus water from the bottom of Lake Montebello, the receiving reservoir of the Gunpowder supply, and if possible allow no water to waste over the weir, and that he should allow the supply from Jones' Falls and its receiving reservoir on Druid Hill to waste as formerly over the weir.

Lake Montebello has a depth of 30 ft. Druid Hill Reservoir has a maximum depth of 65 ft. and a capacity of 495 000 000 galls., and at that time the daily consumption from it was 10 000 000 galls. There are two inlets to the Druid Hill Reservoir, one at the upper end and one about the middle, both of them discharging into the reservoir at the bottom. The outlets from this reservoir are three-quarters of a mile from the upper end and 15, 26 and 39 ft. respectively below the surface.

There was no complaint of bad water until the latter part of November, 1882, nearly two years after the water had been drawn down, when the water in Druid Hill Lake was found bad 50 ft. below the surface and the water in all of the other reservoirs was good, Druid Hill being the only one from which a flow or waste from the bottom had not been kept up during the intervening period.

On February 16th, 1883, Mr. Martin advised the speaker that he found the water in Druid Hill lake good at 5, 10 and 15 ft. below the surface, but bad below that, so it would appear that there had been a

Mr. Darrach. partial overturning, but that the bad water was not brought to the surface.

Attention is called to the necessity of making ample provision for drawing off the old water from the bottoms of storage reservoirs and preventing fresh waters from running to waste, especially when it may be found necessary to conserve the rainfall of the catchment area for a longer period than a year.

Mr. Stearns. F. P. STEARNS, M. Am. Soc. C. E.—The speaker believed that the subject of temperature of ponds and lakes is not only of scientific interest, but that it is of much practical importance that engineers connected with water-works should be familiar with the seasonal changes of temperature, particularly those which occur in deep ponds and either produce or prevent vertical currents at different seasons of the year, thereby having an important influence upon the quality of water at different depths.

In a great majority of deep ponds and reservoirs, the lower layers, which are stagnant during the summer and consequently unable to get any fresh supply of oxygen from the air, accumulate free ammonia and other solid and gaseous products of decomposition, and the water in them becomes more and more offensive toward the end of the season of stagnation. It is therefore desirable, in such cases, that the water for use should be taken from near the surface, and that if there is water to be wasted it should be taken from the bottom layers which contain the greater amount of impurity, since the whole body of water will be improved thereby when the whole of the water in the reservoir is thoroughly mingled by the overturning in the autumn.

There are a few deep ponds and reservoirs in which there is so little decomposition that the oxygen in the lower layers is not used up and the products of decomposition do not accumulate. In such cases a water which is both cold and good can be drawn from the lower layers. The ponds and reservoirs of this kind are somewhat rare, but an instance of a natural body of water is furnished by Lake Winnepesaukee, in New Hampshire, in which the water was found to be as good in the summer, at a depth of 110 ft., as at the surface; and an instance of an artificial body is furnished by Reservoir No. 4, of the Boston Water-Works, in which the bottom water is practically of the same quality as that at the surface. Reservoir No. 4 was carefully prepared for the reception of water by the removal of all the soil and vegetable matter, and from the time it was first used, not only has there been no accumulation of the products of decomposition in the bottom layers, but there has been no excessive growth of algæ or other organisms. This is a record which very few natural ponds can furnish, and is a very important argument for the thorough cleaning of storage reservoirs.

The extent to which the products of decomposition have accumulated is well indicated by the amount of free ammonia, and the follow-

ing table gives a number of instances of the amount of free ammonia found at the surface and near the bottom of various ponds and reservoirs during the period of summer stagnation. Mr. Stearns.

Locality.	Date.	Depth of pond or reservoir. Feet.	Depth of deepest sample. Feet.	Free Ammonia.	
				Surface.	Near bottom.
Jamaica Pond, Mass.....	Aug. 14th, 1890...	57	50	0.0000	0.4720
Waban Lake, Mass.....	Aug. 27th, 1889...	36	35	0.0012	0.1760
Lake Cochituate, Mass.....	Sept. 18th, 1890...	65	60	0.0004	0.0680
Wenham Lake, Mass.....	July 24th, 1889..	46	45	0.0000	0.0560
Boston, Mass., Reservoir 4.....	Aug. 31st, 1887...	46	40	0.0000	0.0021
Lake Winnepesaukee, N. H.....	Aug. 28th, 1893...	..	110	0.0000	0.0000

So far as the speaker was aware, the first information regarding the accumulation of free ammonia in the bottom of ponds during the summer was furnished by Professor William Ripley Nichols, to whom the engineering profession is indebted for much knowledge on the subject of water supply. He was aware that a sample of water collected from near the bottom of a deep pond, in October, 1877, contained a large amount of free ammonia, and he did not agree with the statement of another chemist that this sample probably came from the neighborhood of a mass of decaying matter. In May, 1878, he instituted a series of tests which continued for a year, in which the character and the temperature of the water at the surface, mid-depth and bottom of this pond, which had a maximum depth of 46 ft., was determined frequently.* The observations were made weekly during the summer of 1878, and at less frequent intervals during other portions of the year. He determined the seasonal changes in temperature substantially as now known, and discovered the large accumulation of free ammonia in the bottom layers during the summer.

The subjects of temperature of the water of deep ponds and the effects of temperature upon the quality of water of deep ponds have also received extended treatment in the special report of the Massachusetts State Board of Health upon the Examination of Water Supplies, 1890, pages 659–676 and 749–767.

In determining the character of the water of a deep pond or reservoir which it is proposed to use for water supply purposes, much can be learned by analyzing samples of water taken from the bottom layers during the period of stagnation.

The author, at the end of the paper, makes the following statement: "This uniformity of temperature in ordinary lakes takes place at two points in the yearly profile, and at these times the wind effects are felt

* See Health Supplement of Report of Massachusetts Board of Health, Lunacy and Charity for 1879, p. 100; also, *Transactions* of the American Society of Civil Engineers for 1884, p. 76.

Mr. Stearns. to great depths." The speaker agreed with this statement, but he thought it does not go far enough to give a clear idea as to the great and complete overturning which takes place in the spring and to a greater extent in the autumn. As the surface water cools in the autumn and becomes heavier than the water below the surface, vertical currents are produced which extend down to and somewhat beyond the depth where the water is at the same temperature as at the surface. These currents are nearly continuous and extend deeper and deeper as the season advances until some time in November, when they extend to the bottom of the pond. After they have reached the bottom, they continue to keep the water in motion for several weeks until the whole of the water in the pond has reached the temperature of maximum density.

Observations of temperature at different depths when the water is cooling and overturning indicate that the water in the pond may be overturned several times a day, and analyses of samples from different depths show by their uniformity that the water is most thoroughly mingled.

In connection with this subject of the temperature of lakes, it is interesting to compare the annual curves showing the mean temperature of the water and of the air. Such curves applicable to Massachusetts are shown in Fig. 12.

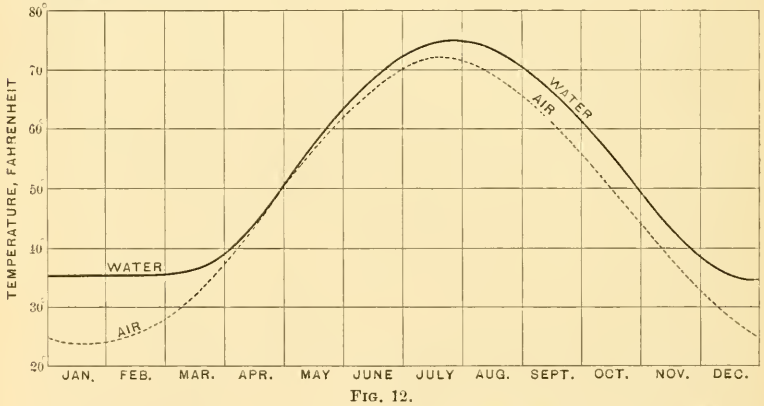


FIG. 12.

It will be noticed at once that the mean temperature of the water is somewhat higher than the mean temperature of the air, although at one point in the spring the two curves coincide. It seems surprising that the temperature of the water in the spring should not be lower than that of the air, since both are rising at the rate of about $\frac{1}{2}^{\circ}$ Fahr. per day, and it would naturally be thought that the air would warm up much more quickly than the water. The reason for the higher mean temperature of the water is not clear, but it seems probable that the direct effect of the sun's heat upon the water has an important influence in producing the results observed.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Note.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

757.

Vol. XXXIV.—August, 1895.

THE PHILADELPHIA AND READING TERMINAL RAILROAD AND STATION IN PHILADELPHIA.

By Joseph M. Wilson, M. Am. Soc. C. E.

Read at the Annual Convention, June, 1895.

WITH DISCUSSION.

Progress in this country is evidenced in no way more conclusively than in the growth of railways. With their great development, their consolidation into grand trunk lines, the concentration of traffic, and the expansion of suburban travel, the necessity has arisen in cities of the first class for great terminal railway stations in central locations, provided with all the improvements, accommodations and facilities for rapid and effective train service which modern ingenuity can devise. It is believed that an account of the construction of one of these great terminals from its inception to its completion will be of considerable interest to the profession, and it is with this object that the present paper has been prepared.

The Philadelphia and Reading Terminal Railroad was constructed, as its name suggests, for the purpose of providing a proper terminal in

the city of Philadelphia for the Philadelphia and Reading Railroad system. The Philadelphia and Reading Railroad was originally built about the year 1838 as a coal road, extending from Pottsville, Pa., to Philadelphia, a distance of 93 miles. By its subsequent extensions and branches and by the acquirement of the control of other roads the company now operates 2 210 miles of railroad, and, excluding the Atlantic City Railroad in southern New Jersey, it has 2 094 miles of railroad terminating in Philadelphia. Its lines extend through or into 18 counties of Pennsylvania which have an aggregate area of 9 805 square miles and a population of 2 400 000.

At Bound Brook, N. J., connection is made with the Central Railroad of New Jersey, furnishing a route to New York City; at Bethlehem, Pa., it joins the Lehigh Valley Railroad and provides direct communication between Philadelphia and the Lehigh and Wyoming Valleys, and also a route to Buffalo and the Great Lakes; at Williamsport, Pa., it unites with railroads leading into the bituminous coal fields of northern and western Pennsylvania; and by its Lebanon Valley division and branches, passing through the rich country of Lancaster, it reaches Harrisburg, the capital of the state, and, after crossing the Susquehanna River, it penetrates the Cumberland Valley toward Gettysburg and goes on to the valley of the Potomac. At Philadelphia a connection is established with the Baltimore and Ohio Railroad, furnishing communication with Baltimore, Washington and the South. This company had four passenger terminals in Philadelphia, two of them legacies from leased lines, all located in inconvenient points far removed from business centers, but by the construction of its terminal road it has now established its main passenger station in the heart of the city and within two blocks of the City Hall.

In order to show justification for the erection of such a terminal as has been built, it may be stated that in 1891 the number of passengers carried by the Philadelphia and Reading Railroad was over 18 000 000, and in 1893 there were over 20 700 000, of which about 10 000 000 arrived at or departed from Philadelphia. About 75% of the latter might be classed as suburban; that is, comprised within a radius of 20 miles from the city. At the present time 294 passenger trains arrive at and depart from the terminal station daily.

The project involved not only the building of a station, but the construction of lines of railroad connecting the old terminals together.

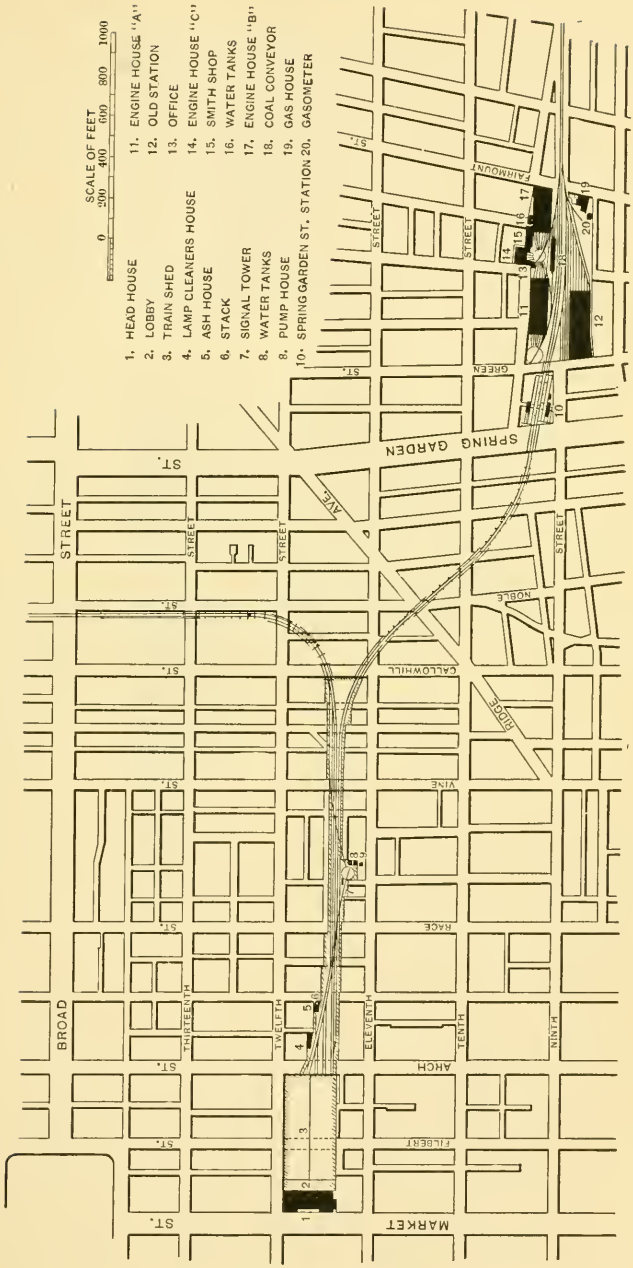

SCALE OF FEET
0 200 400 600 800 1000
1. HEAD HOUSE
2. LOBBY
3. TRAIN SHED
4. LAMP CLEANERS HOUSE
5. ASH HOUSE
6. STACK
7. SIGNAL TOWER
8. WATER TANKS
9. PUMP HOUSE
10. SPRING GARDEN ST. STATION
11. ENGINE HOUSE "A"
12. OLD STATION
13. OFFICE
14. ENGINE HOUSE "C"
15. SMITH SHOP
16. WATER TANKS
17. ENGINE HOUSE "B"
18. COAL CONVEYOR
19. GAS HOUSE
20. GASOMETER
BROAD STREET
THIRTEENTH STREET
TWELFTH STREET
ELEVENTH STREET
AVE.
SPRING GARDEN
GREEN ST.
FAIRMOUNT
NOBLE
CALLOWHILL
VINE
RACE
ARCH
TENTH
NINTH
FILBERT
MARKET
ST.

Fig. 1.

The existing connections in the outskirts of the city made it possible to do this by bringing together two roads, the main line of the Philadelphia and Reading Railroad on Pennsylvania Avenue, with its terminal at Broad and Callowhill Streets, and the Germantown and Norristown Railroad, with its station at Ninth and Green Streets. From these two lines the new terminal railroad was built.

The Terminal Railroad.—The lines of track forming the Terminal Railroad and connecting the station with the previously existing roads consist of a main line and two branches. The main line starts out from the station with 13 tracks which reduce in a distance of 350 ft. to five. It then continues northward to Wood Street, a distance of about 1 650 ft. from the end of the train-shed, where the five tracks separate into two branches of two tracks each. One of these branches turns westward and at a further distance of about 1 600 ft. connects with the main line of the Philadelphia and Reading Railroad at Broad and Noble Streets. The other branch takes a northeasterly direction and about 2 100 ft. beyond Wood Street joins the Germantown and Norristown branch at Ninth and Green Streets.

Active measures were taken in the spring of 1888 to commence the work on the Terminal Railroad, but the natural conservatism of the Quaker City, combined with opposition from rival railroad interests, caused a long delay, and it was not until December 26th, 1890, that a city ordinance was secured granting municipal consent for the construction of the line. The first contract for the road was executed on May 15th, 1891.

The whole of the railroad is built on private property acquired by the company, and from which existing buildings had to be removed. In some cases annoying delays occurred on account of litigation in securing the ground. Fig. 1 shows a general map of the road.

The mileage of the road is as follows:

Miles of first track	1.317
" second track	1.307
" third track	0.467
" fourth track	0.544
" yard track and sidings	2.998
Total mileage operated (all tracks)	6.633

PHILADELPHIA AND READING TERMINAL R.R.

ARCH BRIDGE OVER RACE STREET.

Scale

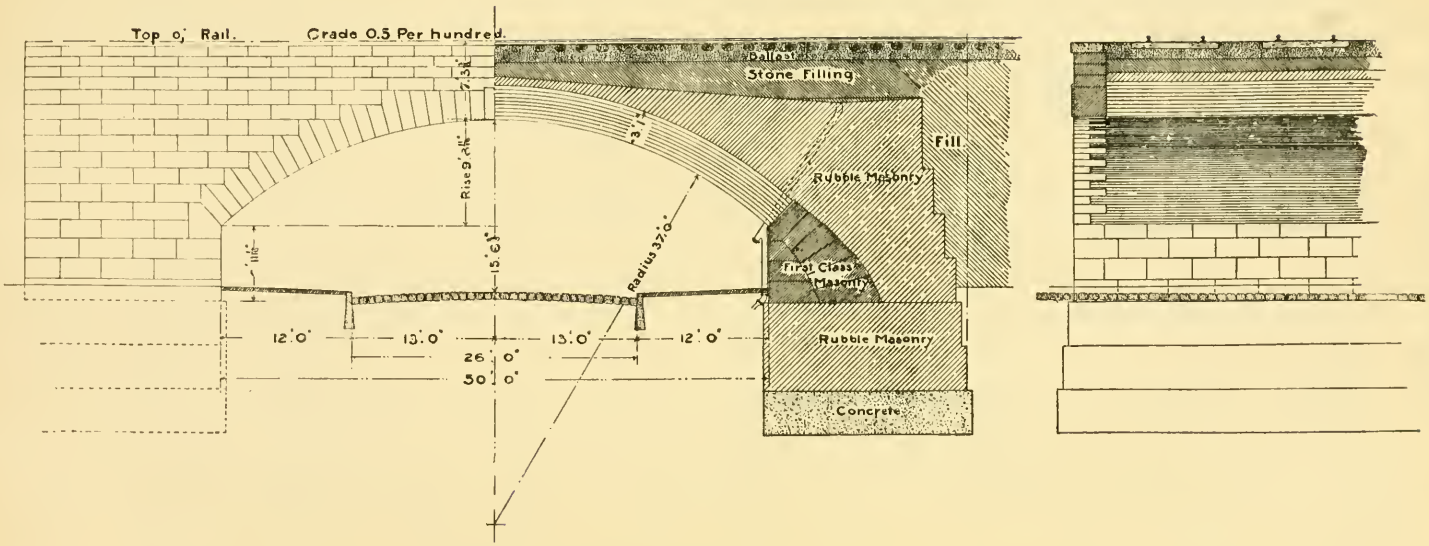


FIG. 2.

The line crosses Arch Street by a plate girder bridge of 72 ft. clear span; thence, between Arch and Cherry Streets, it is carried by a wrought-iron column and plate girder construction over the power-house of the station, and from Cherry to Callowhill Streets, about 1 520 ft. in length, it is laid on a solid roadbed filled in behind retaining walls of stone masonry.

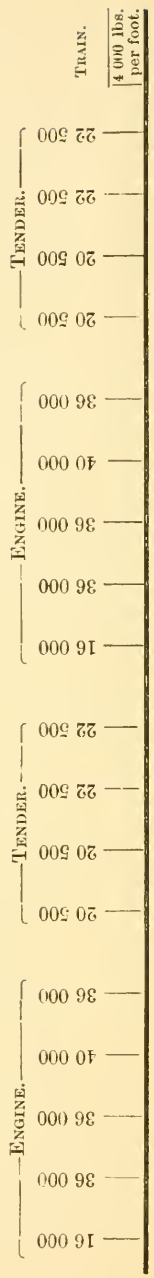
The street bridges on this part of the line are brick arches with stone-face walls. There are two bridges of 50 ft. clear span between skew-backs, two of 40 ft., two of 30 ft., and one of 20 ft., making a total of 260 ft. Fig. 2 shows the bridge over Race Street, and the other bridges are of similar construction. Beyond the south line of Callowhill Street, which is about 250 ft. north of the point of separation of the branch lines, the two branches are carried by wrought-iron elevated girder construction, the branch to Ninth and Green Streets being of the type shown by Fig. 3, with wrought-iron columns, and that to Broad and Noble Streets having stone piers for most of it, although a small portion is also supported by iron columns.

Spring Garden Street is crossed by a skew, pin-connected, open through truss bridge of about 126 ft. 2½ ins. clear span. The other street bridges are plate girders.

The total length of iron elevated structure on both branches, including the bridges over streets, is 2 665$\frac{1}{10}$ ft.

This work was designed to carry two loads, one on each track, trains headed in the same direction on adjacent tracks, as follows:

Two typical engines coupled, weighing with tender 125 tons each, distributed as shown in the diagram and connected with a train weighing 4 000 lbs. per lineal foot of track. The specifications were in accordance with the standard of Messrs. Wilson Brothers & Company, of date April 11th, 1891.



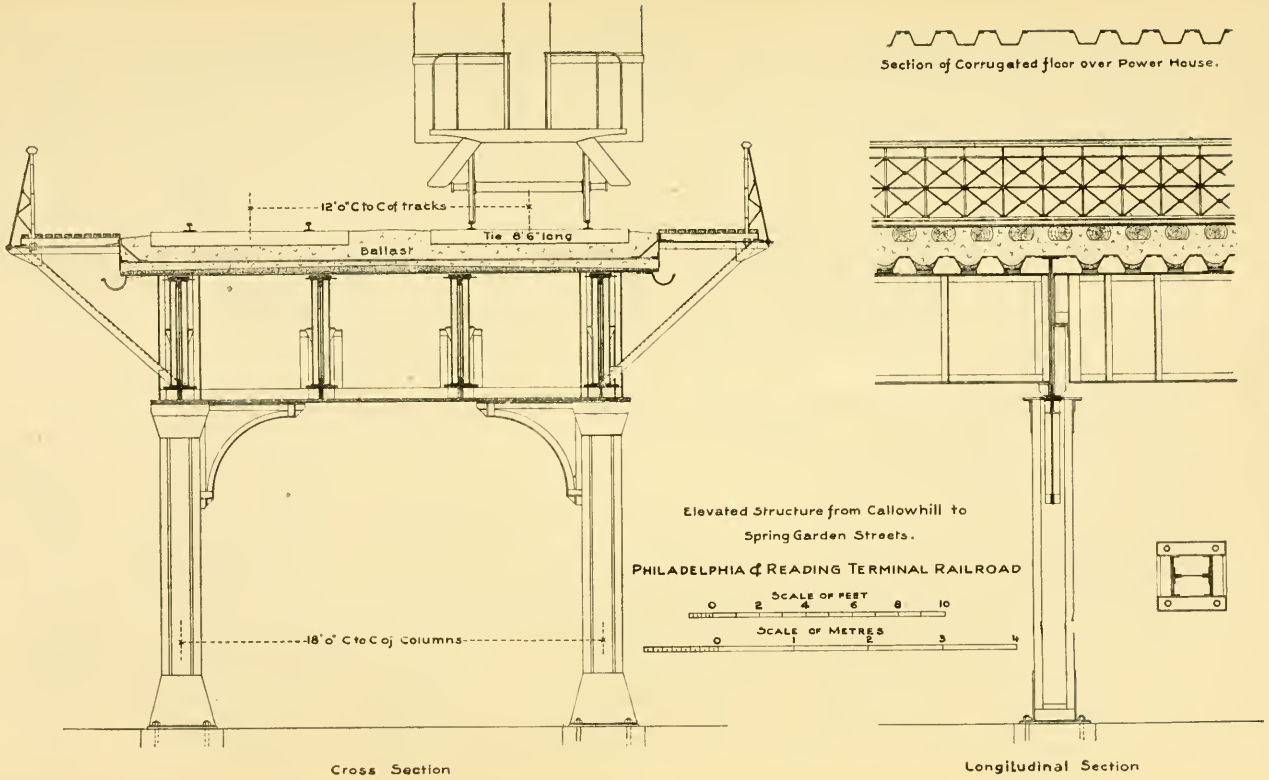

Section of Corrugated floor over Power House.
12'0" C to C of tracks
Tie 8'6" long
Ballast
18'0" C to C of Columns
Elevated Structure from Callowhill to
Spring Garden Streets.
PHILADELPHIA & READING TERMINAL RAILROAD
SCALE OF FEET
0 2 4 6 8 10
SCALE OF METRES
0 1 2 3 4
Cross Section
Longitudinal Section

FIG. 3.

The various contracts made for the station are given as follows, and show the order and rate of progress of the work :

	Contractor.	Date.
Foundations of train-shed	Charles McCaul	July 9, 1891
Structural iron work of train-shed from foundation masonry up to and including track floor	A. & P. Roberts & Co	Aug. 17, 1891
Granite work of train-shed	Charles McCaul	Sept. 5, 1891
Train-shed roof,	Phœnix Bridge Co	Oct. 12, 1891
Train-shed, work up to and including second floor	Charles McCaul	Oct. 20, 1891
Train-shed, structural iron work between north line of Filbert Street and north line of headhouse	A. & P. Roberts & Co	Dec. 29, 1891
Work between Arch and Cherry Streets, iron work	Cofrode & Saylor	Dec. 29, 1891
Floors and paving, granolithic curb	Vulcanite Paving Co	Dec. 29, 1891
Granite work of head-house	Charles McCaul	Jan. 28, 1892
Completion of train-shed and market-house	" "	Feb. 2, 1892
Iron work for stalls and market-house	Wharton Railroad Switch Co	Mar. 2, 1892
Portion between Arch and Cherry Streets	Charles McCaul	Mar. 16, 1892
Wood work for stalls in market-house	" "	Mar. 25, 1892
Work in head-house up to second-floor level	" "	Mar. 30, 1892
Boilers	Edge Moor Iron Co	Apr. 20, 1892
Elevators	Stokes & Parrish Elevator Co	Apr. 21, 1892
Terra cotta work	Perth Amboy Terra Cotta Co	Apr. 25, 1892
Marble work for fish stands	Atkinson & Myhlertz	Apr. 28, 1892
Face brick	Perth Amboy Terra Cotta Co	June 30, 1892
Sheet metal work, copper cornice, etc	J. S. Thorn	July 18, 1892
Refrigerating and cold storage plant	Pennsylvania Iron Works Co.	July 30, 1892
Completion of head-house	Charles McCaul	Aug. 19, 1892
Arc electric lighting	Western Electric Co	Nov. 4, 1892
Burglar-proof vault	Damon Safe and Iron Works Co	Sept. 1, 1892
Floors in restaurant, kitchen, express office and driveway	Vulcanite Paving Co	Nov. 9, 1892
Heating head-house, etc	Onderdonk Heating and Ventilating Co	Nov. 12, 1892
Temporary waiting-rooms	Charles McCaul	Jan. 3, 1893
Ventilating plant for head-house	Onderdonk Heating and Ventilating Co	Jan. 31, 1893
Water supply and drainage, furniture fittings of restaurant-kitchen, bakery, etc	Charles McCaul	Feb. 1, 1893
Wiring power-house for incandescent electric lighting	M. R. Muckle, Jr., & Co	Feb. 7, 1893
Track signs	D. A. Woelpper & Co	Feb. 8, 1893
Gate signs	Albanus L. Smith & Co	Feb. 8, 1893
Three Westinghouse automatic compound engines for power-house	M. R. Muckle, Jr., & Co	Jan. 31, 1893
Cold storage-rooms	Pennsylvania Iron Works Co	Apr. 13, 1893
Direct current incandescent electric lighting	M. R. Muckle, Jr., & Co	June 5, 1893
Wire screens and guards for floor lights, etc	DeWitt Wire Cloth Co	July 12, 1893
Overhead tracking, switches, etc., in exterior corridors and elevators in cold storage vaults of terminal market, etc	J. W. Moyer	July 15, 1893
Tunnel enclosures, wood and glass partitions, iron steps and floors, coal-bin doors and brass railings, etc., in power-house and cold storage vaults	Charles McCaul	July 20, 1893

In describing the station building the various portions of it are considered separately in detail, arranged in such order as has seemed most desirable.

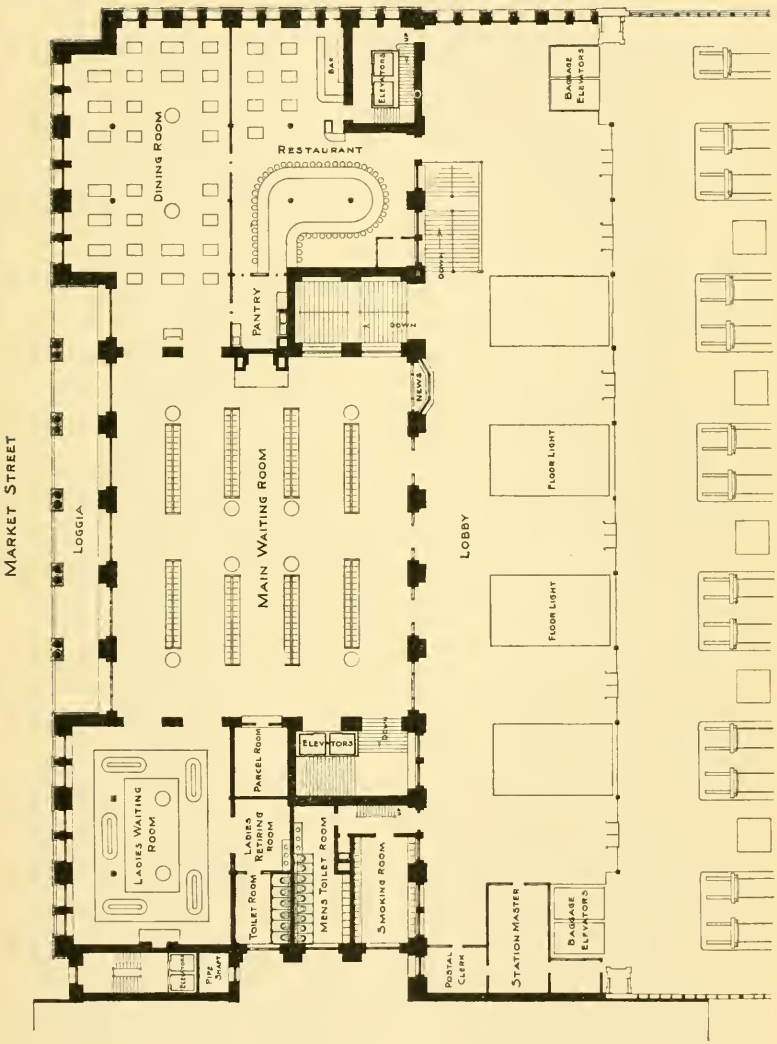

MARKET STREET
LOGGIA
DINING ROOM
RESTAURANT
BAR
ELEVATORS
BAGGAGE ELEVATORS
PANTRY
OPEN
DOWN
NEWS
MAIN WAITING ROOM
LOBBY
FLOOR LIGHT
FLOOR LIGHT
LADIES WAITING ROOM
PARCEL ROOM
ELEVATORS
OPEN
LADIES RETIRING ROOM
TOILET ROOM
MENS TOILET ROOM
SMOKING ROOM
POSTAL CLERK
STATION MASTER
BAGGAGE ELEVATORS
TELEPHONE
PIPE SHAFT

FIG. 4.

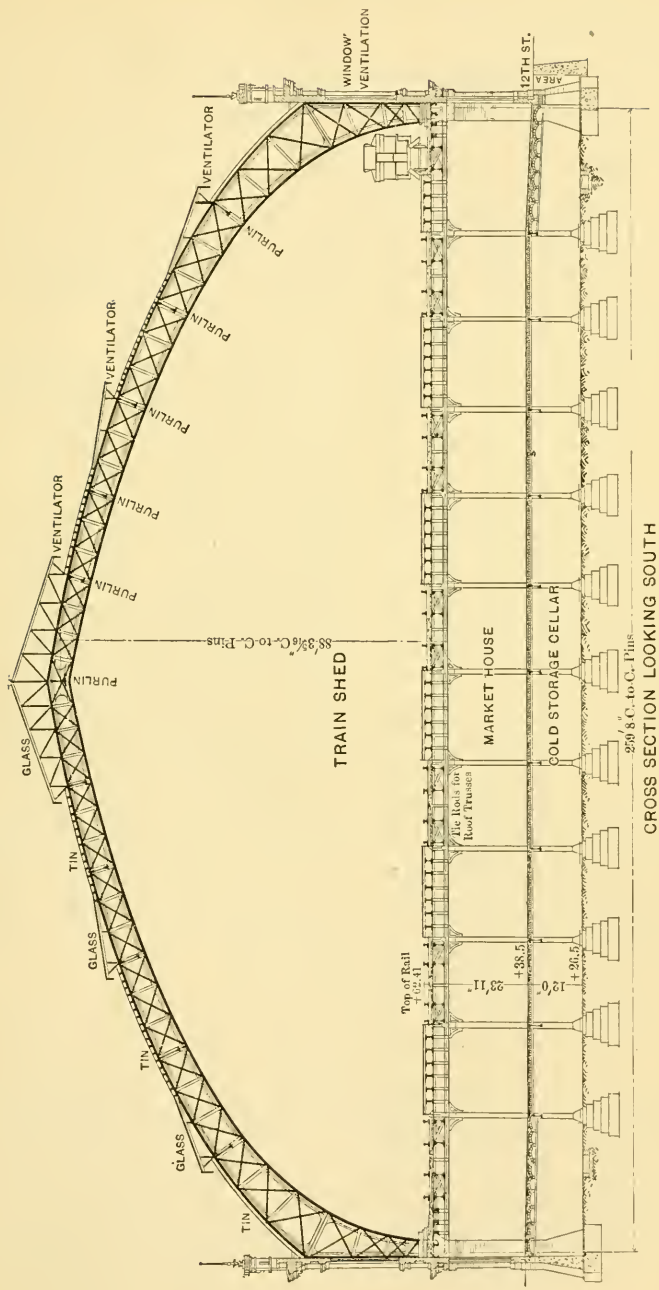
The only steel used in the elevated railroad work was in the lower chord links of the Spring Garden Street Bridge and in the corrugated floor over the power-house. Acid open-hearth steel was allowed, having an ultimate strength of 62 000 to 68 000 lbs. per square inch, an elastic limit of one-half the ultimate, and a minimum stretch in a length of 8 ins. of 20 per cent.

A 60-ft. turn-table is placed on the terminal line about 1 000 ft. north of Cherry Street and two tables of the same size at Ninth and Green Streets, where are also located three brick engine-houses, covering a combined area of about 46 000 sq. ft., capable of housing 40 engines and also providing a small repair shop.

There is also at this point a locomotive supply station, designed and built by the C. W. Hunt Company, of New York, in which four locomotives can simultaneously take coal, sand and water, and dump ashes. All solid material is handled by the Hunt conveyor. Coal and sand are thrown into pockets below the tracks from drop-bottom cars, and are then conveyed for a vertical height of 69 ft. to storage bins, from which they are drawn for supply. Locomotives discharge their ashes also into receptacles below, and the conveyor carries them to a bin in the upper part of the house, from which they may be filled into cars for removal. The capacity of the conveyor is such that coal may be stored at 60 tons, sand at 80 tons, and ashes at 40 tons per hour.

The terminal work also comprised an elevated station at Spring Garden Street, shown on Fig. 1, to serve the local traffic formerly accommodated by the old station at Ninth and Green Streets.

The site selected for the terminal station was on the northeast corner of Market and 12th Streets, with a frontage on the south on Market Street of 266 ft. $10\frac{3}{4}$ ins., and on the west extending back on 12th Street 662 ft. $11\frac{5}{8}$ ins. to Arch Street, the north front on the latter street being 276 ft. $7\frac{1}{4}$ ins. The width of Market Street is 100 ft., that of 12th Street 50 ft., and Arch Street 72 ft. The property was intersected by two cross streets parallel to Market Street, Filbert Street 51 ft. wide and 307 ft. $1\frac{1}{8}$ ins. north of Market Street, and Cuthbert Street 30 ft. wide and 136 ft. $4\frac{3}{4}$ ins. north of Filbert Street. Cuthbert Street was abandoned and closed, but it was found necessary to retain Filbert Street. The property was also accessible by another street, Hunter Street, 30 ft. wide, extending on the east side from 11th Street



CROSS SECTION LOOKING SOUTH

FIG 5

at a point 181 ft. ⅛ in. north of Market Street, but not passing through it. The curb elevations on the street, referred to city datum,* vary from + 36.16 to + 38.09, and the elevation for the top of rail in the station was fixed at + 62.416, with level branches entering the station from the north and crossing Arch Street by a bridge. These fundamental data determined the arrangement of the station and fixed the location of the head-house, facing on Market Street and extending back only so far as necessary to provide the requisite accommodation for waiting-rooms, offices, etc., as much space as possible being allowed between it and Arch Street for the train-shed, with a passenger corridor between it and the head-house.

The elevation of the railway tracks being about 25 ft. above the street grades, required the principal floor of the station to be placed in the second story of the head-house, the first or ground floor being available for entrance, waiting-rooms, ticket offices, baggage-rooms, etc. It having been decided to establish the general offices of the company at the station, certain portions of the first floor, not otherwise required, and all of the floors above the second story, could be appropriated for this purpose. It is evident that there still remained unappropriated a vast extent of valuable space on the ground floor under the train-shed, and the fact that the property between Market and Filbert Streets was occupied by two important market houses, controlling an excellent and well-established business, which were to be demolished for the erection of the station, suggested the idea of constructing a new market house on the ground floor under the train-shed and of transferring the valuable business of the old markets to it. It was also decided that cellars could be built under the market for cold storage, and that such an arrangement would possess particular advantage in allowing a ready application of refrigeration for the preservation of market supplies, thereby dispensing with the use of ice.

A large space under the entrance tracks on the north side of Arch Street, having a frontage of 149 ft. 2⅝ ins. and extending back 285 ft. 7¼ ins. to Cherry Street, was found available for the operating plant of the terminal. By the continuation of Hunter Street through the station as a private way, a space was secured for the delivery of baggage and for use as a cab-stand. These are the principal features

* City datum is 8.732 ft. above the level of the Atlantic Ocean at Raritan Bay and 2.24 ft. above mean high tide at Philadelphia.

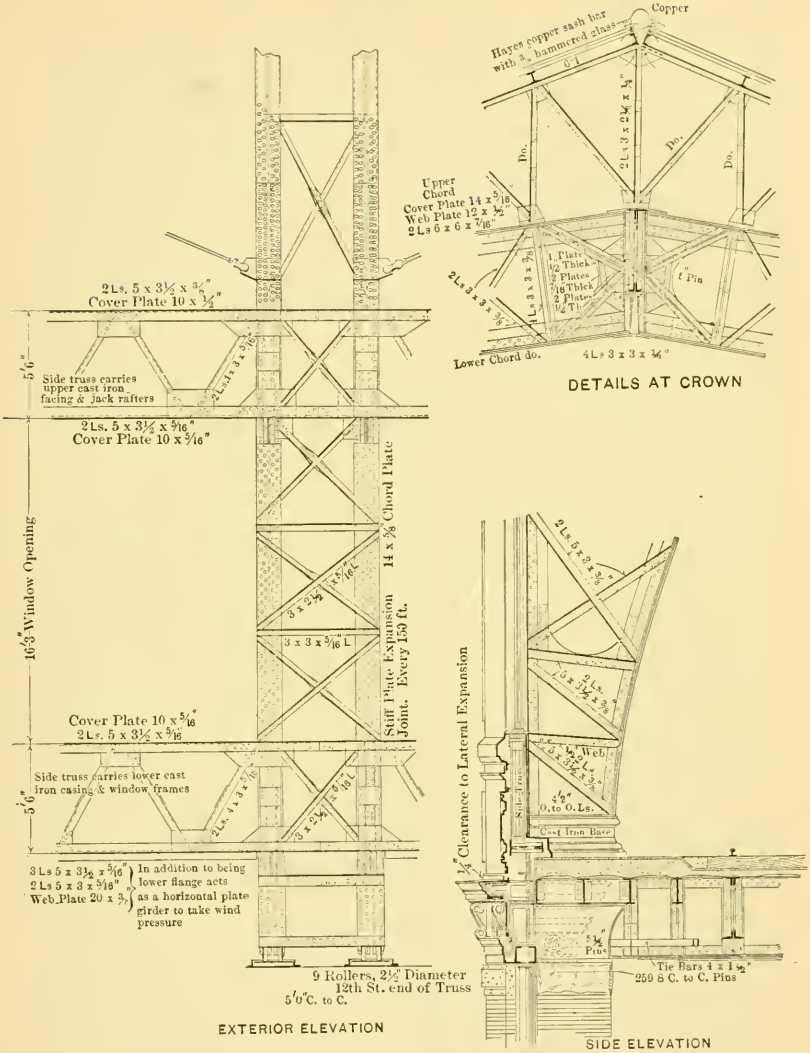
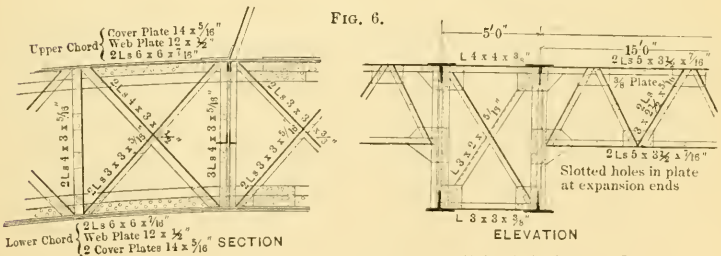


FIG. 6.



CONNECTION OF PURLIN WITH MAIN TRUSS

FIG. 7.

which governed the arrangement of the plans for the station. Possession was obtained of that portion of the property between Filbert and Arch Streets in advance of that south of Filbert Street, and the necessity of pushing the work to completion as rapidly as possible, together with the great advantage that would accrue from having the new market house built before the old one was demolished, decided that the work on the station should be executed by successive steps in a series of contracts, beginning at Arch Street and working south, the head-house being the last portion of the structure completed.

The various complications arising from these circumstances and also incident to the construction of a railroad through a solidly built-up city made the time required for the entire work longer than it would have been had the ground been cleared in advance. The work was, however, successfully carried out in accordance with the programme adopted, the new market house was built, the tracks for the train-shed laid and a water-proof floor put down; the occupants of the old markets were transferred without interruption to business, and the construction of the head-house proceeded with on the old site. The tracks and train-shed were opened for travel (using temporary passenger rooms) on January 29th, 1893, and the permanent passenger rooms in the head-house were completed and put into use on October 16th of the same year.

The Head-House.—A number of preliminary sketches and designs were prepared by the engineers and architects, Messrs. Wilson Brothers & Co., and after their careful consideration it was decided to adopt the Italian renaissance as the type of architecture for the structure. In reaching this decision, the president of the company availed himself of the services of Mr. Francis H. Kimball, of New York (who was at that time doing certain work for the Philadelphia and Reading Railroad Company), as a "consulting architect on the exterior ornamentation." Mr. Kimball also made some preliminary sketches and drawings for the exterior treatment, acting in unison with Messrs. Wilson Brothers & Co. in this matter, and the final design for the exterior of the head-house as prepared by that firm and adopted was the resultant of such consultation and Mr. Kimball's suggestions. All of the general drawings and the scale details of the ornamentation were prepared by Messrs. Wilson Brothers & Co.; the full-size drawings of the terra

cotta details and the inspection of the terra cotta at Perth Amboy were done by Mr. Kimball.

The face of the head-house is set back 6 ft. $6\frac{1}{8}$ ins. from the building line of Market Street, so as to increase the width of the pavement in front, and the depth of the structure on Twelfth Street is 100 ft. 6 ins. On the first floor, back of the head-house proper, are the baggage-rooms, which occupy an additional depth of 73 ft. $11\frac{3}{8}$ ins., and cover the space all the way to an open passage through the station from Hunter Street to Twelfth Street, intended for cab service and access to baggage-rooms by wagons.

In designing a building of this kind and extent, the proper mode of procedure is first to determine a certain unit of square or rectangle by which the ground plan can be divided up into a number of parts, and to the intersecting or division lines of which the columns, walls, partitions, etc., may be made to conform, although it is possible that in special cases some variation from these lines may be necessary. In the present case the key to the arrangement of plans is a unit rectangle of 16 ft. north and south by 20 ft. east and west as may be seen by reference to the drawings.

The building consists of a cellar, basement, and practically eight stories, the second and third stories being merged into one high story in that portion of the second floor occupied by waiting-rooms. The central portion of the first story is appropriated to passenger entrances, ticket offices and waiting-rooms, while the east and west portions are used for offices; and as these parts do not require as high ceilings as the waiting-rooms, the floors are raised, allowing basements to be placed under them.

The elevation of first floor in central portion is.....	+ 39.75
" " " at east and west ends is.....	+ 44
" " basement floor is.....	+ 32.5

A cellar is constructed under the whole of the head-house, the elevation of the floor being + 22.5, thus giving under the central part of the building a height of 17 ft. 3 ins. from floor to floor, and 10 ft. under the basement ends.

The heights of the several stories of the building, measured from floor to floor, are as follows:

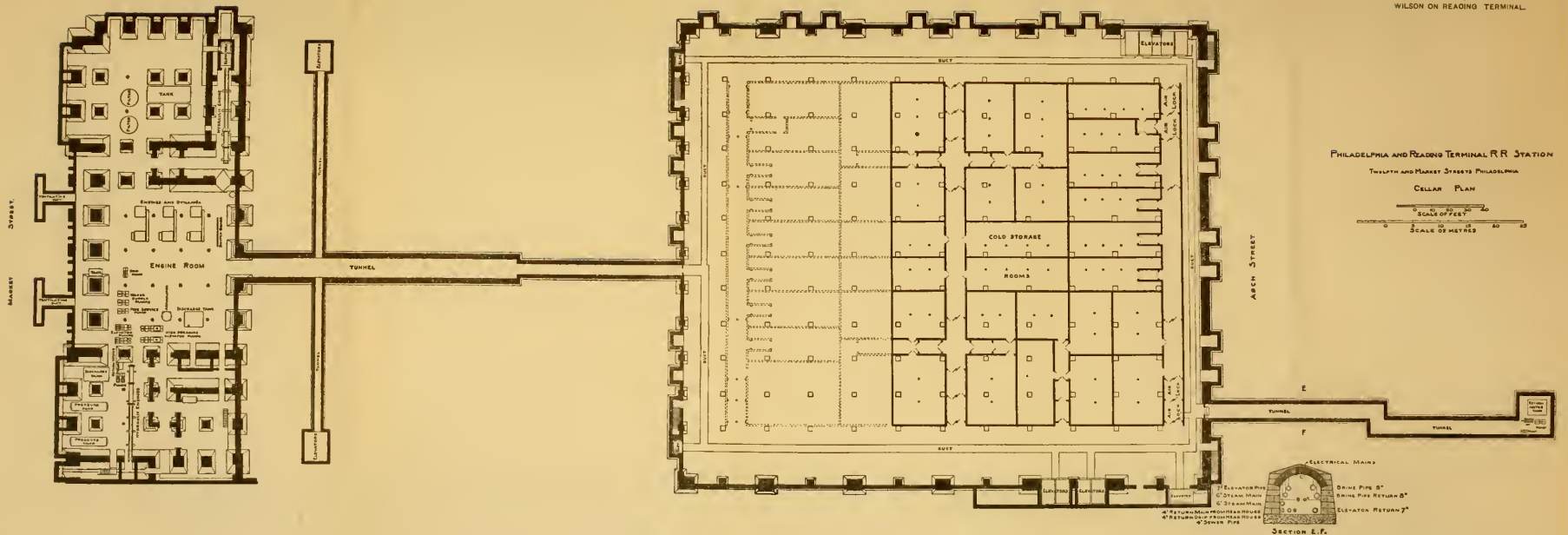
	Feet.	Inches.
Cellar to basement	10	0
" " first floor where no basement exists	17	3
Basement to first floor over basement	11	6
First floor to second floor where no basement exists.	23	3
First floor to second floor where basements exist	19	0
Second floor to entresol (or half-story)	19	0
Entresol to third floor	16	0
Second floor to third floor, where no entresol exists.	35	0
Third to fourth floors	16	0
Fourth to fifth "	13	6
Fifth to sixth "	13	6
Sixth to seventh "	13	6
Seventh to attic "	13	6
Attic floor to apex of roof	14	0½
Total height, cellar floor to apex of roof	169	6½

The exterior faces of the building on the street fronts are constructed of pink eastern granite for the height of the basement and first story, and above that of pink brick and cream-white terra cotta. For facing the rear elevation and the light well, a pale red brick, known as telephone brick, is used. The upper part of the cornice of the building is of copper.

The construction of the head-house, from the foundation up to and including the second floor, is fireproof, having wrought and cast-iron columns, wrought-iron or steel beams, and brick floors. For a portion of the work Gustavino flooring is used. Above the second floor a slow-burning construction has been adopted, consisting of iron or steel columns and beams, with heavy plank floors; the columns, beams and undersides of floors being protected with mackite plasterboards on which the finishing plaster is laid. The partitions are also constructed of solid mackite, 6 ins. in thickness, a material that, while very light in weight, is, from its composition, principally plaster of paris, asbestos fiber, and encased reeds, unexcelled in its fire-resisting properties. The roof is of the same general construction as the upper floors of the building, and is covered with tin.

While wrought-iron columns are used to support the main girders which span the waiting-rooms, yet the construction of the exterior

PLATE I.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXIV, No. 757.
WILSON ON READING TERMINAL.



PHILADELPHIA AND READING TERMINAL R.R. STATION

TWELFTH AND MARKET STREETS PHILADELPHIA

CELLAR PLAN

SCALE OF FEET

SCALE OF METRES

walls of the building is not based on a skeleton iron or steel system of building, and columns are not used in these walls except for the support of the special main girders above mentioned. The thickness of exterior wall on Twelfth Street, which represents an average for the building, is as follows:

	Feet.	Inches.
Concrete foundation	16	
Base of masonry resting on concrete	14	
First story	5	6
Second story to third floor	5	
Third and fourth stories	4	
Fifth and sixth stories	3	6
Seventh story and attic	3	

It must be observed that the walls are merely a series of piers, the windows being deeply recessed for architectural effect, and the curtains under the windows very much reduced. This accounts for what might otherwise appear to be an excessive thickness.

Cellar.—The cellar provides ample room for all of the operating machinery needed in this part of the building, and a main tunnel, running north to the cold storage cellar, and from that continuing under Arch Street, connects it with the power plant which is placed on the property north of that street (see Plate I). This tunnel has a clear cross-section of 11 ft. width by 11 ft. height to crown of arch, as far as Hunter Street, and 6 ft. 2 ins. width by 7 ft. 1 in. height beyond that point to the cellar of the market house, which it enters with its bottom 2 ft. below the cellar floor. There is no basement or cellar under the baggage-rooms, but cross-tunnels 4 ft. wide by 6 ft. clear height run from the main tunnel to the locations of the elevators in these rooms. The cellars have cement floors.

Basements.—The basements under the east and west ends of the building are each divided into three rooms, with entrances from the street by steps down from the pavement, so that they may be rented as stores. The rooms average 750 to 1 000 sq. ft. of floor surface each, and they have wooden floors. Convenient lavatory and sanitary arrangements are provided.

First Floor.—There are two entrances on the first floor communicating with the office floors in the upper part of the building, one on Market Street and one on Twelfth Street, each provided with a stair-

way and two elevators. The main passenger entrance is in the center of the Market Street front, and exits from the second floor passenger corridor are provided by stairways on Market and Twelfth Streets.

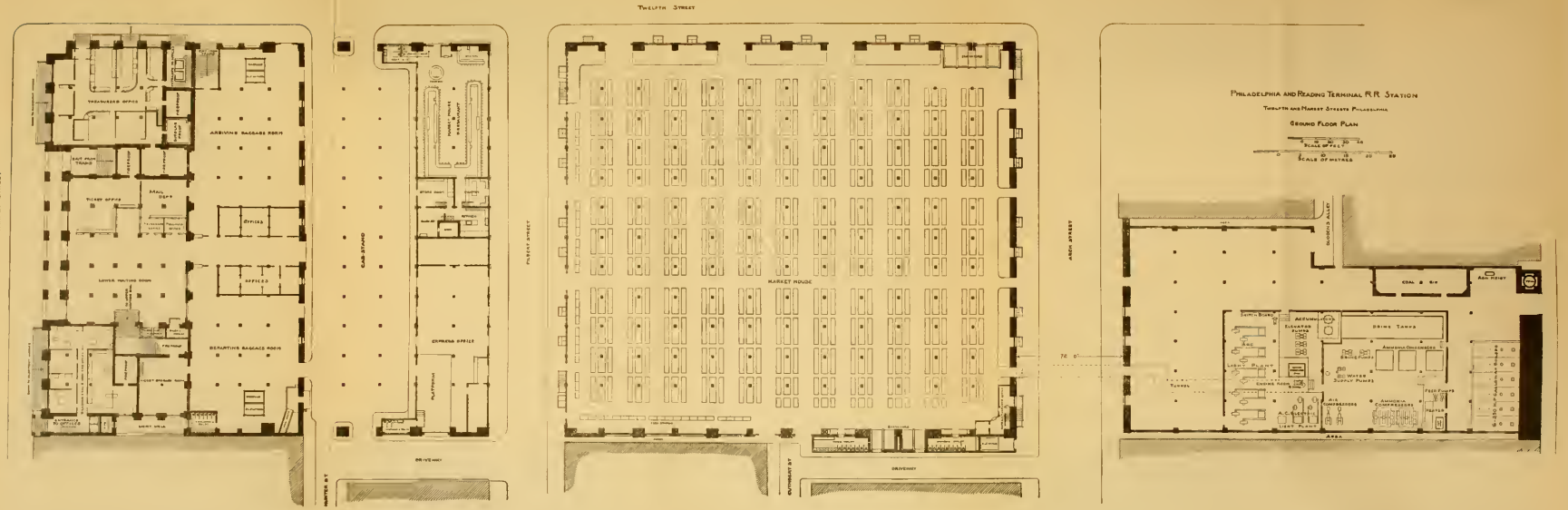
The central part of this floor is occupied by the waiting-room, ticket office, sub-post office, Pullman and telegraph offices; the western part, at the corner of Twelfth and Market Streets, by the Treasurer's office, and the eastern portion, fronting on Market Street, by the Coal and Iron Department, back of which are the ticket storage-rooms. To the rear of the building are the baggage-rooms with a corridor 20 ft. in width between them, running from the waiting-room to the Hunter Street extension cab-stand. On each side of this corridor are arranged a series of offices, cut off from the baggage-rooms and averaging 240 to 400 sq. ft. of floor area each, those on the right being intended for the medical, missing freight and baggage departments, while those on the left are for the company's mail and advertisement inspectors (see Plate II).

Two large passenger elevators and a stairway communicate from the first to second story waiting-rooms, and each baggage-room has two elevators of sufficient size to carry baggage trucks to the train-shed floor. The Treasurer's office, the ticket office and the Coal and Iron Department each have ample fire and burglar-proof vaults. The sub-post office has a mail chute from the second floor so as to afford facilities to passengers for mailing letters. The floors of the main waiting-room and baggage-rooms are of cement, and all the other rooms, used as offices, have wooden floors. At the east end of the building is a ventilation and light area 45 by 12 ft., and a shaft about 7 ft. 6 ins. by 10 ft. 8¾ ins. for carrying supply pipes, steam pipes, etc., to the top of the building.

The floor areas of the various rooms in this story are approximately as follows:

	Square feet.
Passenger waiting-room	5 055
Ticket office, exclusive of vaults	1 391
Sub-post office	1 116
Coal and Iron Department, exclusive of vaults	2 830
Ticket storage-room, together with floor in half story above	2 442
Treasury office, exclusive of vaults	4 791
Incoming baggage-room (western room)	7 094
Outgoing baggage-room (eastern room)	6 308

PLATE II.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXIV, No. 757.
WILSON ON READING TERMINAL.



Second Floor.—On the second floor will be found a large general waiting-room occupying the central part of the building with a loggia opening out from it and overlooking Market Street (see Fig. 4). On the east of this room, fronting on Market Street, is the ladies' room with adjuncts, and on the west is a large dining-room, to the rear of which is a restaurant. Back of the ladies' room and accessible from the general waiting-room is the news-room, and still further back are the gentlemen's smoking-room and lavatories opening on to the passenger corridor. Access is gained by a special stairway to a barber shop in a half-story over the lavatories.

The floor areas covered by the several rooms on this floor are :

	Square feet.
General waiting-room	7 857
Ladies' "	2 552
Ladies' toilet-room, etc	585
News-room	300
Gentlemen's smoking-room	493
Gentlemen's lavatories, etc	555
Dining-room	3 570
Restaurant	2 676
Serving-room	287

The floors are of marble, except that for the central portion of the ladies' waiting-room a parquet floor is used. Dumb-waiters connect the serving-room with the kitchen department in the top story of the building. The open porch or loggia is a delightful feature of the general waiting-room, extending on the Market Street front for the whole length of the room and affording an opportunity for those necessarily detained at the station waiting for trains to pass the time pleasantly in observing the life and activity in the street.

The Entresol.—East and west of the main waiting-room and between the second and third floor is placed a half-story which is divided into office rooms for the operating department of the terminal, a communication being made between the two portions by means of a gallery in the main waiting-room.

Upper Floors.—The upper floors of the building are used for the general offices of the Philadelphia and Reading Railroad Company, the President's suite being placed on the third floor in the southwestern

part, and having a pleasant feature in a circular oriel window in the recess at the street corner. Contiguous are the rooms for the Board, Secretary, etc. The corridors or hallways on these floors are very spacious, being 16 ft. in width, and the rooms generally have about 20 by 30 ft. floor area, although in some cases a number of rooms are thrown together and in others they are divided up into smaller sizes, depending upon the uses to which they are appropriated. The construction of the building is such that partitions can be placed to suit the wants of the occupants, wherever needed. The upper portions of all partitions on the corridors are formed of glazed sash so as to furnish ample light to the corridors.

The kitchen department for the dining-room and restaurant is placed in the attic and includes a general store-room, pantry, refrigerator, dish-room and serving-room. The tanks for the storage of the water supply for the building are also placed on this floor. Of these there are eight; six of them are each 11 ft. wide, 18 ft. long and 3½ ft. deep, and two, 8 ft. wide, 18 ft. long and 3½ ft. deep, the total capacity at 2½ ft. depth of water being about 27 600 galls. Five of these tanks are for rain water collected from the roof, and three for filtered water. They are connected by a system of main and distributing pipes so that they can be used in unison, separately, or cut off, as may be desired. Photographic rooms are provided in the attic with roof skylight and ample facilities for blue-print copying, with eight printing frames each 48 by 60 ins. Large fire-proof vaults are provided on each floor, built up with brick from the foundations.

Passenger Lobby.—On the second floor between the head-house and the train-shed is the great passenger corridor or lobby, 50 ft. wide and extending in length for the whole width of the station. At the east end of this lobby will be found the office of the station-master and a small room for the postal clerk. Near the west end a wide exit stairway descends to Twelfth Street, and close by is another to Market Street. The floor of the lobby is of cement. Four large floor lights, each of 20 by 32 ft. area, light the baggage-rooms in the story below.

The Train-Shed.—Beyond the passenger lobby is the train-shed, with its great three-centered, pointed arch roof in a single span for the whole width of the tracks, and hinged at the crown and springing line (see Fig. 5). The dimensions of this arch are as follows :

	Feet.	Inches.
Span, center to center of pins at springing	259	8
Height between centers of pins at crown and springing	88	$3\frac{5}{16}$
Total span out to out of building line	266	6
Height from center of pin at springing to starting point of outer curve at eaves	33	$4\frac{1}{2}$
Depth of arch rib at crown	6	0
Pin at crown is placed in center of depth of arch rib	..	..
Depth of arch rib at haunch	12	$4\frac{3}{4}$
Depth of arch rib at springing	4	0
Center of pin at crown to top of ventilator	14	0
Height from pavement to center of pin at springing	20	6
Extreme height from pavement	122	$9\frac{5}{16}$
Length of train-shed, center to center of end trusses	501	8
Width of lobby	50	0
Total length, including head-house measured to building lines	662	$11\frac{5}{8}$
Diameter of pin at crown of arch	..	5
Diameter of pins at base	..	$5\frac{1}{2}$

The exterior curve of each semi-arch has a radius of 120 ft. for an angular distance of $41\frac{1}{2}°$ from the horizontal, and thence a radius of 310 ft. to the crown, while the interior curve has a radius of 50 ft. for an angular distance of $39\frac{1}{2}°$ from the horizontal; thence a radius of 85 ft. for a further angular distance of 22°, and thence a radius of 280 ft. to the crown. The arch is therefore a compound curve, four-centered for the outer flange member and six-centered for the inner flange. Its form and proportions were designed with great care, so as to produce what was judged to give in its outlines the best artistic and architectural effects, and at the same time agree with the requirements of economical construction.

To the architectural engineer the roof is always an interesting study, and when roofs of large span are to be considered, the interest proportionately increases. It is not merely that the space is to be covered with a substantial roof, but the architectural question of an artistic design in harmony with the other parts of the building is of first importance.

A handsome roof is the refinement of engineering construction. The author, some years ago, designed a number of three-hinged arched roofs of the same general type with pointed arch, as in this roof, for the Pennsylvania Railroad Company, one for the Pittsburgh passenger station in 1867 of 158-ft. span, one for Washington in 1872 of 110-ft. span, and one for Jersey City, also in 1872, of 220-ft. span, but they were not adopted and erected on account of the supposed unwarranted expense. Now it has come to be recognized that this is the proper form of roof for a large railway station, reducing to a minimum the destructive action to the iron or steel construction from the sulphurous vapors emitted by the locomotives, and adding essentially to the comfort and satisfaction of travelers by increased ventilation and improved æsthetic effect.

In designing such a roof it is an object to mass the material together as much as possible, avoiding a great number of small pieces and leaving wide open spaces; also to provide ample light and ventilation.

It will be noticed that the arches are arranged in pairs, placed 50 ft. 2 ins. apart, center to center, those of each pair being 5 ft. apart to centers. The spacing brought two pairs of arches over Filbert Street, this street being 51 ft. wide, and the center line of each pair is 5 ins. beyond the house line of the street and the end of the wall. The weights from these arches are carried by wrought-iron girders acting as cantilevers with projecting stone brackets below.

The assumed loading for computations was taken as follows:

	Pounds per square foot.
Weight of trusses, purlins and jack rafters	25
" covering	8
" snow	12
Total	45

Wind pressure was assumed at 35 lbs. per square foot against a vertical surface, the pressure normal to the roof surface being, for the first three panels covering an angle of 41° from the horizontal, 30 lbs. per square foot; for the next six panels comprising an angle of 22°, 20 lbs. per square foot; and for the remaining nine and a half panels of 17°, 15 lbs. per square foot.

The roof was calculated for:

First.—Snow on one side and dead load.

Second.—Snow on both sides and dead load.

Third.—Wind on one side and dead load.

Fourth.—Snow and wind on one side; snow on the other side and dead load.

Calculation was made for the main tie rod with wind alone on the roof, to ascertain if it would always remain in tension.

The roof is constructed of wrought iron, under detailed specifications of the engineers and architects, all that used in the tensile members of open trusses, laterals, pins, bolts, etc., being required to be double rolled after and directly from the muck bar (no scrap allowed), and capable of sustaining an ultimate stress of 52 000 lbs. per square inch on a full section of test piece, with an elastic limit of 26 000 lbs. per square inch and a minimum stretch of 20%, measured after breaking in a length of 8 ins. All other wrought iron was required to be tough, fibrous, uniform in quality throughout, free from flaws, blisters and injurious cracks, and with a workmanlike finish, capable of sustaining an ultimate stress of 48 000 lbs. (excepting for plates over 24 ins. in width, where the requirement was 46 000 lbs.) per square inch on a full section of test piece, with an elastic limit of 26 000 lbs. per square inch and a minimum stretch for that used in tension or transverse stress (except web plates) of 15 per cent. For that used in compression and for web plates not exceeding 24 ins. the minimum stretch was fixed at 10%, and for web plates over 24 ins. at 5 per cent.

The working stresses were taken as follows: Tension, 14 000 lbs. per square inch; compression, 12 000 lbs. per square inch, with a deduction for column length. The calculations were made graphically.

Two intermediate jack rafters of single intersection triangular type are placed between the main arches, and carried by the main purlins, for the purpose of supporting secondary wooden purlins, on which the sheathing is laid. Lateral ties are introduced between the two adjacent arches at each end of the roof, and on each side of the structure throughout its whole length from the eaves up to the first main purlin. The main purlins divide the roof on each side into six panels and are single intersection triangular trusses. The secondary purlins are of yellow pine, 4 by 6 ins., in section, placed about 2 ft. 4 ins. to centers, where the roof is not glazed, and on them is laid $\frac{7}{8}$-in. tongued

and grooved yellow pine sheathing boards, covered with tin. Details of the roof are shown in Figs. 6 and 7.

There is one central ventilator on the ridge of the roof and four side ventilators, all of them glazed, giving a width of glass across the roof of about 126 lin. ft., independent of the glazed gable ends and the windows in the side walls. This large amount of glass, together with the simple design of the roof, the material being massed together in as small a number of pieces as possible, with very few cross-rods breaking up the spaces and disfiguring the main lines of construction, gives an abundance of light and produces a remarkably pleasing effect.

The exterior walls of this part of the building, from the foundations up to the bearing plates of the arches, have concrete foundations, then stone to the ground line, above which they are of brick with pink granite base-course and caps. Above the springing of the roof arch, the side walls are of cast-iron and glass, on wrought-iron framing. The interior finish up to the tops of the windows is of corrugated iron and above that of yellow pine, tongued, grooved and beaded. The wrought-iron framing for the side walls is attached to the arched trusses, and on the side where the rollers of the arches are placed, the wall is cut horizontally just below the track level, the upper portion being suspended from the arches and given ½-in. clearance from the lower part to allow of movement when the arches contract or expand.

Those parts of the gable ends of the roof not glazed are finished in copper, as are also the pinnacles on the pilasters of the side walls. The interior ironwork is painted a shade of Indian red, touched up with black, and the woodwork is finished to show its natural color. The whole effect is extremely pleasing, and much more satisfactory than the usual lead paint finish which so soon discolors from the action of sulphur gases.

There are 13 tracks in the building, arranged in pairs, except for the eastern track, and the passenger footways are 20 ft. wide to curbs, except the eastern one, which is 13 ft. wide.

The floor of the train-shed is constructed of buckled plates carried by plate girders. A layer of concrete is first placed over these plates and on this 1 in. of Neuchatel asphalt, the ties for the tracks being laid in asphalt concrete and the footway pavements formed of Neuchatel asphalt, the top being 8 ins. above the top of rail. Large

frames filled with heavy hammered glass are built in the footways at frequent intervals to provide additional light to the market on the first floor.

The Market.—The whole of the first floor between Filbert and Arch Streets is occupied by the Reading Terminal Market, covering an area of about 78 000 sq. ft., with an average clear height to the underside of main girders of the floor above of 18 ft. The side walls are filled with large windows occupying almost the whole area between the pilasters, and there are also frequent doors opening on to the streets. The floor of the market is formed of brick arches laid in cement and carried on iron beams, the finish on top being of Neuchatel asphalt. In much of this floor, north of the head-house, cement would have been preferred, but it had to be done in the winter season, and in a limited time; hence the necessity of using asphalt. The walls and piers are of brick faced with buff pressed brick.

The market provides for about 1 260 stands (see Plate II), and the stalls are designed on the most approved plan, with iron frames and white oak counter-tops, rails, divisions and lockers, except for a certain number of the stalls and for the fish market, where the counter-tops are of heavy slabs of white marble. Convenient toilet rooms are provided for men and for women. There are eight hydraulic elevators running from the cold storage in the basement to the main-track floor over the market for the transfer of market produce, meats, etc., four of these being located near the corner of Twelfth and Arch Streets and four on the east wall. There is also an elevator on the east wall contiguous to Arch Street, running from the level of the market floor to the cold storage floor.

Market House Restaurant.—The western portion of that part of the first floor between Filbert Street and the cab-stand is occupied by a market house restaurant, with store-rooms, pantry, kitchen and bakery, covering in all an area of 126 by 48 ft., while the eastern part of the same location is arranged as an express office, with a floor area of about 6 250 sq. ft. The whole of the area under the market is excavated and forms a basement which is occupied by cold storage rooms, the clear height of this basement being about 11 ft., and along Twelfth, Arch and Filbert Streets are numerous ventilation areas, 18 in all, extending from the floor up to the pavement level. There are four stairways from the street, two on Arch and two on Filbert Streets; also two

small elevators on Filbert Street in addition to those already mentioned in connection with the market house.

The Power-House.—This building, located under the railway tracks between Arch and Cherry Streets and containing the operating plants for the cold storage and arc electric light system, and also the boilers, covers a space about 285 ft. in depth, 141 ft. in width on Arch Street and 114 ft. on Cherry Street (see Plate II). It is plainly and serviceably constructed of brick and stone, and does not require special description, the tracks overhead being carried by columns and girders. These tracks are laid on stone ballast on a corrugated floor, the corrugated steel pieces forming this floor being of a special section, which, it is believed, has never previously been employed. It was essential to obtain a water-proof floor, and the usual shapes of material would not furnish this on account of the positions of the riveted connections. It was therefore decided to use U-shaped troughs with horizontal outward flanges on the upper edges, and to connect adjacent pieces by a cover plate laid on top and riveted to the contiguous flanges (see detail in Fig. 3). Some of the manufacturers bidding on the work claimed that this shape of piece could not be made, but it was successfully done by stamping cold from $\frac{3}{8}$-in. steel plates in 20-ft. lengths, at no greater cost than if the floor had been built with the usual shapes. The steel work was painted with P and B paint, and the hollows or troughs were partially filled and sloped for drainage with a composition consisting of a mixture of coal tar and fine gravel, which became very hard. The drainage is carried from these by a series of galvanized iron corrugated pipes to main drains, hung from the superstructure, and thence to sewers. The work has been a complete success, giving very dry and satisfactory rooms below for the purposes required.

Space is provided at the north end of the house for six boilers, and the stack is at the northwest corner. It is of brick, having a height of 150 ft. above the floor level of the boiler-room and an internal diameter of 7 ft., the base being square, changing to an octagon above, and finishing at the top with a neat cast-iron cap. The core of the stack, which is entirely separated from the external portion, is 13 ins. thick at the lower part for a height of 60 ft., then 9 ins. for a further height of 30 ft., and above that $4\frac{1}{2}$ ins. The exterior is 2 ft. 6 ins. thick at the square base, with 2 ft. 6 ins. additional on the interior

corners, and the octagonal portion is 22 ins. in thickness to a height of 60 ft. above the floor, thence 17 ins. for 30 ft., thence 13 ins. for 30 ft., thence 9 ins. to the top. A large coal bin is provided, which is filled directly from cars on a siding track above, and a hydraulic hoist is arranged so that ashes may be raised and automatically dumped into cars for hauling away. The positions of the cold storage and the arc-lighting plants are shown by the drawings.

The Operating Plant of the Station.—The operating plant comprises the boilers, the heating and ventilation apparatus, the electric plant for the lighting of buildings and roadway, the pneumatic switch apparatus, the elevator machinery, the water supply and the refrigerating apparatus. The entire plant has been so arranged as to allow as much economy in working as possible under existing conditions, and at the same time avoid risk of stopping the operation of the road and station from accident.

The boilers, refrigerating machinery, pumps, and accumulators operating the freight elevators, the air compressors for the switch system, the arc-light dynamo and engines, and the alternating-current incandescent light dynamos and engines are located in the power-house between Arch and Cherry Streets (see Plate II). The direct-current incandescent dynamos and engines, the filters, tanks and pumps for supplying water to the station, the pumps and compression tanks, hydraulic engines, etc., for the passenger elevators, a relay of pumps and an accumulator for the freight elevators, and the pumps, etc., to discharge waste water to the sewer, are located in the basement under the central portion of the head-house on Market Street (see Plate I).

Steam from the boilers is delivered into two 10-in. steam mains, each boiler being connected to each main by 7-in. pipes, and these mains in turn feed the steam supply pipes, of which there are four of 6 ins. diameter, each pipe being connected to each of the two 10-in. mains in the boiler-room. One of these 6-in. pipes supplies the brine pumps of the refrigerating apparatus, the pumps serving the freight elevators, and the engines driving the arc dynamos. Another serves the engines driving the ammonia compressors and the engines of the alternating electric-light dynamos, feed-water and sewage pumps. Supply is also provided from both of these pipes to the air compressors of the switch plant, and the two pipes are so connected that either

one or both can be used for all of the machinery in the power-house. The other two 6-in. steam supply pipes carry steam a distance of 1 200 ft. through the tunnels and cold storage cellar to the head-house engine-room, where they form a loop and supply steam to the engines, pumps, etc., placed there.

The pipes in the power-house are hung from the girders forming the roof covering and carrying the tracks above. In order to avoid danger of accident from vibrations due to movement of locomotives and trains, these pipes are suspended by chains, and the result has been entirely satisfactory, no leaks or trouble of any kind having been experienced in connection with them.

The exhaust steam from the engines and pumps in the Cherry Street power-house is carried by pipes laid in ducts under the floor to a horizontal feed-water heater, with a by-pass so arranged that the tubes of the heater may be removed or repaired without interference with the operation of the plant. The exhaust main from the feed-water heater is 18 ins. in diameter and runs under the boiler-room floor and is carried up outside the stack. The exhaust steam from the engines and pumps in the head-house is also carried by pipes laid in ducts under the floor to the vertical pipe shaft, and up this shaft by a main exhaust pipe 18 ins. in diameter to the attic, and thence by 12-in. pipe to above the roof, where it terminates in an exhaust head. Below this exhaust head a balanced back-pressure valve is placed, so arranged as to open with a pressure of 1 lb. above the atmosphere. Under this valve a 16-in. branch supplies the mains in the attic with the steam required for heating the buildings.

The water of condensation from the heating system is discharged into a tank and pumped back to the feed pumps and thence returned to the boilers. The exhaust steam which passes through the feed-water heater is wasted, and this deficit is made up by the water which runs from the ammonia condenser coils of the refrigerating plant. During the winter the condensation of steam used in heating the head-house is at a maximum and furnishes a large amount of the water returned to the boilers when the refrigerating plant is but lightly taxed. During the summer the increased amount of water from the refrigerating plant makes up the deficiency in the condensation from the head-house, so that the supply to the boilers is constantly kept up and no water is bought for steaming purposes, all heat possible being utilized.

Boiler Plant.—The boiler plant consists of six Galloway boilers, aggregating 1 200 H. P., two feed pumps, four injectors, one horizontal tubular heater, a blow-off tank, a hot well and a hydraulic ash lift. The Galloway boiler was adopted for this location principally on account of the low head-room available, and also because such boilers are readily cleaned, and the character of the water which has to be used is such as to produce large quantities of lime scale.

Each boiler is 7 ft. in diameter and 28 ft. $1\frac{3}{4}$ ins. in length, set as shown on Plate III in an air-tight and well insulated brick chamber, 28 ft. $4\frac{1}{2}$ ins. long by 8 ft. wide by 10 ft. 6 ins. high, inside dimensions, the top of the boiler being placed within 3 ins. of the intrados of the crowning arch. A space of $5\frac{1}{2}$ ins. is left between the shell and the walls of the chamber at the springing line of the arch, and 2 ft. at the back of the boiler. The boiler is supported at its front end in the brick setting and 10 ft. from the rear on a cast-iron expansion rocker. The setting is of hard brick, 18 ins. in thickness, laid in Portland cement, except where it comes close to the hot gases, when lime mortar is used, and a wrought-iron casing of No. 22 B. and S. gauge is built in the wall to prevent air leakage, a brick baffle being constructed to protect the rocker. The spandrels between the arches and the space above them are filled with concrete, the top being finished with a horizontal cement pavement; the exposed walls are faced with white enameled brick, and the front is capped with a course of dressed bluestone. Two cleaning doors, 15 ins. in diameter, are set in the front wall and below the boiler shell. Each boiler has two furnaces, 2 ft. 10 ins. in diameter by 6 ft. 7 ins. long, provided with Tupper grate bars for anthracite pea coal. The boiler shell is made up of seven rings of $\frac{1}{2}$-in. best mild steel, each formed of one plate the full circumference of the boiler, the longitudinal seams being triple riveted and alternate, so as to avoid a continuous line of rivets. The heads are each flanged from one plate $\frac{9}{16}$ in. thick and secured to the shell by the flanges, gusset plates and angle irons. The furnaces are of $\frac{3}{8}$-in. plate, and provision is made for expansion.

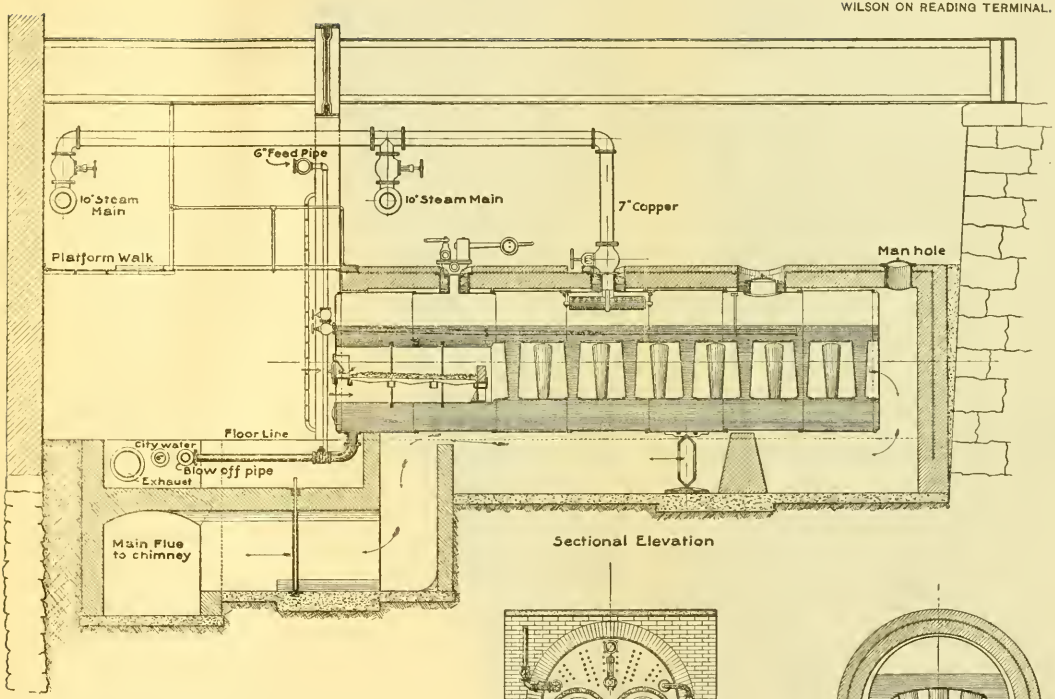
The boiler chambers are lined with fire brick, and each furnace has two suitable fire-clay bridge wall blocks. The furnaces unite behind the fire bridges into one oval flue or combustion chamber, which has large corrugations and contains the cone tubes. The metal in the oval flue is $\frac{1}{2}$ in. thick.

There are in each boiler 33 Galloway cone tubes, which are the essential features of this boiler, each tube being welded and gauged from one plate of $\frac{3}{8}$-in. mild steel and having an internal diameter of $10\frac{1}{2}$ ins. at the upper end and $5\frac{1}{2}$ ins. at the lower end. The tubes are all of the same length and interchangeable. The gases from combustion pass from the furnaces, through the combustion chamber, against the cone tubes and corrugations of the interior shell, and return, enveloping the exterior shell to the damper placed below and under the boiler front, thence to the main flue under the boiler room floor.

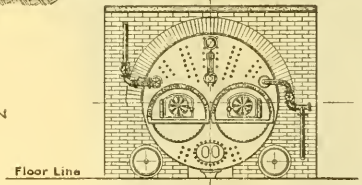
The specification for boilers required the best open-hearth mild steel, free from all cracks and flaws, stamped to show the brand of steel and name of makers, and standing the following physical tests: To have an ultimate strength of not more that 60 000 lbs, nor less than 50 000 lbs. per square inch; an elastic limit of not less than 30 000 lbs. per square inch; a ductility of not less than 25% measured in a length of 8 ins., and a reduction of area at point of fracture of not less than 50 per cent. The test pieces were required to bend cold without crack or flaw 180° to a curve having a radius equal to half their thickness, after having been beaten red hot and quenched in water at 60° Fahr. A variation in cross-section or weight of plates of more than 5% from that specified was cause for rejection.

Each boiler is provided with a manhole 17 ins. in diameter; two nozzles, 7 ins. in diameter; one 2-in. nozzle for surface blow-off; one 4-in. nozzle for blow-off connection; one 2-in. feed nozzle; anti-priming pipe; one 4-in. lever and one 4-in. pop safety valve, and one 7-in. main steam valve; also gauges, valves, dampers, etc. Each boiler has areas, etc., as follows:

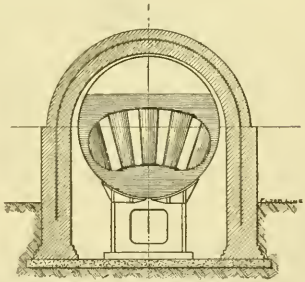
Grate surface			36.25	sq. ft.
Water-heating surface in shell		418 sq. ft.		
" "	fire-box	141 "		
" "	oval flue	200 "		
" "	tubes	201 "		
			960	sq. ft.
Steam-heating "	shell		199	"
Total surface			1 159	"
Steam space			211	cu. ft.
Water "			482	"



Sectional Elevation



Front Elevation
Single Boiler

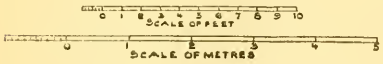


Cross Section

PHILADELPHIA AND READING TERMINAL R.R. STATION

TWELFTH AND MARKET STREETS, PHILADELPHIA

GALLOWAY BOILERS



The boilers were tested under a hydrostatic pressure of 150 lbs. per square inch for a working steam pressure of 110 lbs. per square inch. The tests for capacity and economy developed the following facts, red ash anthracite pea coal being burned, with a natural draft by the stack, 150 ft. in height.

First Test:

Equivalent evaporation from and at 212° Fahr. per hour	9 485.9	lbs. of water.
Total evaporation (including water in coal) from and at 212° Fahr. per hour	9 563.7	" "
Equivalent evaporation per pound of dry coal from and at 212° Fahr. per hour.	8.13	" "
Equivalent evaporation per pound of combustible from and at 212° Fahr. per hour	9.74	" "
Moisture in steam	0.51	per cent.
Percentage of ash in coal	16.50	"
Dry coal per hour per square foot of grate.	32.44	lbs.

Second Test:

Equivalent evaporation from and at 212° Fahr. per hour	8 027.7	lbs. of water.
Total evaporation (including water in coal) from and at 212° Fahr. per hour	8 051.3	" "
Equivalent evaporation per pound of dry coal from and at 212° Fahr. per hour.	8.34	" "
Equivalent evaporation per pound of dry combustible from and at 212° Fahr. per hour	9.90	" "
Moisture in steam	Dry.	
Percentage of ash in coal	15.78	per cent.
Dry coal per hour per square foot of grate	26.63	lbs.
Average steam pressure absolute	106.0	"
Temperature of boiler-room	88°	Fahr.
" back of boiler	680°–810°	Fahr.
" at damper	600°–680°	"

Average test of gases during 10 hours. Parts in 100, by volume:

Carbonic acid	11.58
Oxygen	5.42
Carbonic oxide	0.20
Nitrogen	82.80
	100.00

Approximate weight of air per pound of coal, 16 lbs.

The samples of gas were taken under the various conditions of firing, by which an excess of oxygen might be produced from open furnace doors, or an excess of unconsumed gases from fresh coal being put on the fires, etc.

The feed-water to the boilers can be supplied either from the city water mains, the condensation from the heating systems of the buildings, or the overflow from the ammonia condenser of the refrigerating apparatus. It is delivered into an iron tank, from which it is taken by two Blake duplex feed pumps, having 10-in. steam cylinders, 6-in. water plungers and 12-in. stroke, each pump being capable of supplying all of the six boilers. From the pumps the water passes through a horizontal tubular feed-water heater of the reservoir type, 19 ft. 6 ins. long and 54 ins. in diameter, containing 49 lin. ft. of 2-in. seamless drawn brass tubes, measuring 650 sq. ft. of heating surface, and is carried by a 6-in. main to the boilers, with a temperature, after leaving the heater, of 212° Fahr. The capacity of the heater is 1 890 galls. A further auxiliary supply is obtained from four No. 8 Sellers fixed nozzle automatic injectors, each having a capacity of 13 500 lbs. of water per hour. Each boiler is supplied from the 6-in. feed main by a 2-in. feed pipe, this pipe being carried through the front head and discharging under the surface of the water near the rear of the boiler. The blow-off pipes are 3 ins. in diameter, and discharge through a 4-in. main blow-off pipe, laid in a duct under the floor and above the main flue, to a muffler in a blow-off tank. This tank is 23 ft. 10 ins. long and 5 ft. 2 ins. in diameter, made of ½-in. wrought iron and provided with an 18-in. manhole, a 6-in. vapor pipe and a 6-in. outlet to a sewer. It is located under the roadway to the boiler-room, near the stack.

The main flue is 133 ft. long to center of stack, and has an area at its smaller end of 19 sq. ft., and at the larger end, at the stack, of

40.4 sq. ft. The inlets for each boiler have an area of 14.6 sq. ft. This flue is built of hard brick and Portland cement mortar, is under the boiler-room floor, and above it is the duct, previously mentioned, for the blow-off pipe and for the exhaust main from the feed-water heater.

Each boiler is provided with an individual damper, and a main damper, operated by an automatic regulator, is placed in the flue at the stack. The boilers are controlled entirely from the front, the rear of the boiler-setting abutting against the Cherry Street Bridge abutment. A light iron gallery at the level of the top of the boilers extends the entire length of the 10-in. steam mains, and is connected to the top of the boiler setting, three iron ladders giving access to this gallery for the ready manipulation of valves, etc.

The coal bins, previously mentioned, are of brick, and extend from the boiler-room floor to the track level. They have a siding directly over them, from which coal is dropped into them. They are capable of holding 450 tons. An ash-lift is provided, operated by hydraulic power from the accumulators, and automatically dumps the ashes into cars at the track level. It has a capacity of 51 cu. ft., and the bucket drops into a pocket level with the floor of the boiler-room, allowing easy deposit of ashes.

Heating.—The head-house is heated by direct radiation throughout. The exhaust steam from the machinery in the basement is carried by an 18-in. main up the pipe shaft to the attic, where it is connected to a system of horizontal pipes running around the exterior walls in a loop. Supply pipes controlled by valves are carried from these horizontal pipes by a bottom connection to the various radiators on all the floors of the building above the second. The basement, first and second floors are served from a system of pipes in the basement, connected with the exhaust main, these supply pipes also serving as bleeders for the supply from the attic. All of these pipes are drained to traps discharging into the main drip from the heating system. The condensation from the radiators flows into vertical risers, thence to the main drip in the basement and cellars, which discharges into traps from which it is taken to a 300-gall. drip tank, constructed of $\frac{3}{16}$-in. iron and provided with manhole, water-gauge, vapor pipe, and necessary outlets.

A duplex pump with a capacity of 40 galls. per minute delivers the

water from the drip tank through a 2½-in. pipe to the hot-well in the Cherry Street power-house. This pump is operated automatically by the height of water in the drip tank, and a relay pump is also provided in case of accident. Air outlets controlled by stop and check valves are provided on the drip pipes and discharge into the vapor pipe. The individual parts of the heating system are controlled by gate valves on the steam and drip pipes. The rooms of the building with north and west exposure have 2 sq. ft. of radiator surface, and those with south and east exposure 1 sq. ft. to each 100 cu. ft. of content.

The main supply riser is proportioned to give a sectional area of 0.007 sq. in., and other steam supply pipes 0.012 sq. in. to each square foot of radiating surface, no pipes being allowed of less than ¾-in. diameter.

The drip pipes are proportioned as follows:

1-in. pipe for	150 to	300	sq. ft.	radiating	surface.
2-in. "	600 to 1 600	"	"	"	
3-in. "	3 200 to 7 000	"	"	"	

The intermediate sizes are proportioned accordingly.

Provision is made for a supply of live steam controlled by a pressure regulator, should the exhaust from the machinery in operation be insufficient. In practice this is only occasionally necessary in the early morning. The steam pressure required in the heating system varies from 1 lb. above the atmosphere to 5 ins. of vacuum.

The baggage-rooms, restaurant and toilet-rooms in the market house, the pipes serving nozzles for warming cars in the train-shed, and pipes for clearing rain conductors of ice or snow, are supplied from the live steam mains and controlled by pressure regulators, the drips from these heating surfaces and from the traps draining steam mains, etc., being discharged by a pipe into a drip tank at the end of the tunnel at Arch Street, from which it is pumped into the hot-well.

Ventilation.—The rooms and offices above the second floor are ventilated by exhaust fans located on the roof, and the various air conductors are of galvanized iron, being proportioned to convey the air at a velocity of 600 ft. per minute. Two 60-in. Blackman fans are provided for this purpose, each rated at 12 000 cu. ft. per minute. There is also one 42-in. Blackman fan of 9 000 cu. ft. per minute, specially provided for exhausting the air from the large toilet-rooms in the

eastern end of the building. Each fan discharges its air above the roof by a galvanized iron tower, designed to prevent adverse air currents.

Individual toilet-rooms are ventilated by aspirating coils in flues, these coils being supplied with steam from the high-pressure pipe serving the kitchen department and controlled by pressure regulators. The vent flue registers are located in the side walls of the various rooms above the baseboard, and those from the toilet-rooms are connected to the seat vents of the Sypho water-closet hoppers, each hopper having an air vent of 12 sq. ins.

The 60-in. fans are driven by 5 H.-P. electric motors, and the 42-in. fan by a 2½ H.-P. motor.

The basement stores are ventilated from the floor by galvanized iron ducts and a 30-in. Huyett & Smith disk fan, driven by a 1 H.-P. electric motor, the air being delivered at a rate of 6 000 ft. per minute at the base of the pipe shaft, which has on top a 30-in. Star ventilator.

Arc-Lighting Plant.—The arc-light plant has a total capacity for operating three hundred 2 000 nominal candle-power arc lamps in 10 circuits, with a maximum of 30 lamps on each circuit. Two hundred and forty lamps were installed under the contract, distributed as follows:

66 lamps in the train-shed on two circuits, and from the circuit lighting the streets and cab lobby.

44 lamps in the market house, on two circuits.

12 lamps in streets and driveways around buildings.

28 lamps on line of road between Arch and Callowhill Streets, on five circuits.

48 lamps on branch line to Broad and Noble Streets and beyond Broad Street, on two circuits.

12 lamps on branch line to Ninth and Green Streets, on three circuits.

38 lamps in station at Spring Garden Street, Ninth Street yard and engine houses, etc., on three circuits.

The lamps in the train-shed are suspended over each platform, at distances apart of 50 ft., with the arc 16 ft. above the floor. The conductors run across the shed, 28 ft. above the floor, and are stayed over the center of each platform from the front to the rear of the shed with phosphor-bronze wires of No. 12 B. and S. gauge. These lamps in the

train-shed have full opal glass globes, thus giving perfect illumination without shadows. The lamps on the line of the road are erected on poles, with the arc placed 30 ft. above the track level, and between Arch and Callowhill Streets are spaced generally 100 ft. apart on each side of the road. The lamps are located so as to also illuminate the streets passing under the road. On the branch lines, the lamps are located on the right of the outbound track, at distances apart of about 125 ft.

The tracks are served by five circuits, as already stated, but the lamps are so connected that two of the circuits may be cut out and yet leave a sufficient number of lamps in service to operate the road.

The poles for the road lines are of Michigan pine, 5 ins. square at the top and 10 ins. square at the level of roadway, those between Arch and Callowhill Streets being set either in wrought-iron pedestals or secured by iron clamps to the masonry, and those on the branch lines set 7 ft. in the ground, on large flat stones and well tamped. The poles are dressed and painted in three coats of white lead and oil, and are provided with iron ladders. The conductors are carried up the poles in moldings, and each lamp is provided with a cut-out switch in a cast-iron box with Yale lock, set 4 ft. above the track, and so arranged as to cut out the lamp without breaking the circuit of the other lamps. All of these lamps on the roadway have clear glass globes.

The electrical conductors go out on the right and return on the left side of the roadway, those between Arch and Callowhill Streets being laid in 1¼-in. wrought-iron pipes, either on the coping or under the surfaces of the ground. Brass slip joints are provided for expansion and contraction, and cast-iron junction boxes at each lamp. On the iron elevated construction the conductors are carried under the footways, on glass insulators. All lamps are suspended independently from the conductors.

There are 10 direct current automatic Western Electric arc dynamos, each having a capacity of 30 10-ampere 50-volt lamps, at a speed of 1 400 revolutions per minute, and guaranteed to operate with a power not exceeding ¾ H. P., delivered at the dynamo pulley for each lamp burning. The dynamos are operated by five Westinghouse compound engines (one engine driving two dynamos), connected up with Schultz endless leather-link belts. This type of belt is used throughout the

plant. Each engine has a 10-in. high-pressure cylinder, an 18-in. low-pressure cylinder and a 10-in. stroke, and at 320 revolutions per minute, with 100 lbs. steam pressure, and one-fourth cut-off, it is guaranteed to deliver 65 H. P.

The switchboard is of Tennessee marble and is fully equipped with spring jacks, plugs, ampere meters, lightning arresters, magneto bell, volt meter, etc.

The lamps are 2 000 candle-power of two carbon, Western Electric manufacture. All conductors are of No. 4 B. and S. gauge; those in the underground conduits have simplex triple-braided double extra mining insulation, and those for all other work, triple braided weather-proof insulation.

Incandescent Electric Light Plant.—The incandescent electric light plant is served by both alternating and direct current dynamos. The alternating current apparatus is located in the Cherry Street power-house and serves the lights in the power-house, the market offices and toilet-rooms, the cold storage rooms, the tunnels and basements, the kitchen, restaurant, express rooms, station-master's office and relay lamps in the passenger corridor; also two 500-light (equivalent to 1 000 16 candle-power lamps) converters in the head-house engine-room.

There are two 45-kilowatt Westinghouse alternating current incandescent dynamos with separate exciters driven from the engines, each exciter having a capacity of 30 amperes at 125 volts, the primary current being 1 000 volts and the secondary 100 volts. Each dynamo is driven by a compound Westinghouse engine with a 12-in. high-pressure cylinder, a 20-in. low-pressure cylinder and 12-in. stroke, rated at 100 H. P., with 100 lbs. steam.

In addition to the various circuits serving the lamps as indicated, a pair of No. 4 B. and S. mains is run to the 1 000-light converters which are connected with the head-house distribution switchboard and act as a relay to the direct current incandescent dynamos, as will be hereinafter described.

The direct current incandescent lighting apparatus serves all lamps in the head-house, the motors driving the ventilating fans and the transformers serving current for all the telegraph lines centering in the building. There are three Westinghouse 75-kilowatt, multiple, direct current incandescent light dynamos, each of 600 amperes, at

125 volts. Each of these dynamos is driven by a Westinghouse compound engine, having a 13-in. high-pressure cylinder, a 22-in. low-pressure cylinder and a 13 in stroke, and of 125 nominal horse-power, with 100 lbs. of steam (see Plate I).

The switchboard is of white marble, 18 by 9 ft., supported by brass pedestals. It has two pairs of bus-bars, one pair for alternating current and the other for direct current. The 36 circuits serving the building are each provided with a double pole, double throw, jaw switch, and four spare switches are supplied for additional circuits that may be needed. Each circuit is furnished with two cut-out switches on each pole, so that in the event of the failure of a fuse a second fuse can be thrown into circuit by the closing of the auxiliary switch. Each of the circuit switches is connected to both sets of bus-bars, so that when thrown up they feed the circuits from the direct current bus-bars, and when thrown down from the alternating current bus-bars.

There are provided upon the switchboard for each dynamo its appropriate switches, rheostat, ammeter, volt meter, and upon the top of the board an ammeter indicating the load on the alternating current dynamos in the power-house. This arrangement of the switchboard is somewhat novel, and was so designed as to utilize the spare power of the alternating current machines and avoid the necessity of a spare direct current dynamo.

There are also provided four revolving transformers of different sizes, taking the current at 100 volts from the switchboard and transforming it to 6 volts, 20 volts and 70 volts respectively, for the operation of all the telegraph lines in this building belonging to the road and its connections.

From the fact that the station was put into service before the work was completed, it was impossible to make a test of the engines of the electrical plant, but a test made at the Drexel Institute of similar engines, with 10-in. and 18-in. cylinders and 10-in. stroke, gave the following results. The test was made during a run of five hours after the engines had been in commission for several months. The conditions for the tests were not as favorable as they might have been, as all of the water supplied to the boiler (of 200 H. P. capacity) during the test was weighed and charged against the engine, the steam having been delivered through long lines of large mains.

Test No. 1, *Full Load.*

Engine rating	65 H. P.
Steam	98.1 lbs.
Atmosphere	14.72 "
Horse-power, mean, high-pressure cylinder	33.47
" " low-pressure cylinder	30.19
Total	63.66 H. P.
Water per horse-power per hour	26.08 lbs.
Revolutions per minute	320
Variation	314 to 329

Test No. 2, *Partial Load.*

Engine rating	65 H. P.
Steam	96.2 lbs.
Atmosphere	14.72 "
Horse-power, mean, high-pressure cylinder	24.49
" " low-pressure cylinder	16.75
Total	41.24 H. P.
Water per horse-power per hour	28.79 lbs.
Revolutions per minute	320.5
Variation	320 to 322

There are installed in all, including current for motors, an equivalent of 4 200 16 candle-power lamps. For general illumination one 16 candle-power lamp is allowed for 1 000 cu. ft. of content, which of course does not include special desk lighting. The entire wiring is run on the two-wire system. No wires less than No. 14 B. and S. gauge are used, and wires of No. 8 and larger sizes are stranded. The insulation is as follows: Simplex double extra mining for high tension alternating currents, primary circuits. For the secondary circuits, Bishop double coat. For the direct current incandescent wiring fire and and weather-proof insulation is used.

The general layout is figured for 16 candle-power 100-volt lamps, on a basis of 1.2 amperes per lamp, with a loss not exceeding 5 volts, and a difference of 1% between any two lamps on the same circuit, with all lamps burning.

The conductors are run in brass armored conduits for all rooms below the third floor, but above that they are run in molding, this method having been adopted to enable the ready change in position of lamps, as might be required to suit the various office requirements.

The attic is served with one circuit, and the third to seventh stories inclusive are each served with two main circuits, the feed wires being carried in vertical cabinets, one in the center of the eastern half of the building and the other in the western half. Each main feed wire serves a "crib," run on the ceiling of the central corridor, from which taps are made with link-fuse cut-outs. Each stairway has its own circuit, which also carries lamps for the ends of the corridors. Circuits for the remaining lamps are arranged so as to admit of easy control by the engineer, and the switches and cut-outs in those portions of the building open to the public are set in metallic boxes, flush with the walls, and provided with Yale locks.

Pneumatic Switch and Signal Apparatus.—The pneumatic switch and signal apparatus was installed by the railroad company and is operated by two No. 7 Ingersoll-Sergeant Drill Company's straight-line compressors, each having a 14-in. steam cylinder, a 14½-in. air cylinder, and an 18-in. stroke, the fly-wheel being 5 ft. in diameter. One compressor, with 100 lbs. of steam and working at 36 revolutions per minute, is sufficient to run the apparatus. Connections are made with two steam mains to each compressor, and the electrical storage batteries are charged by an 80-ampere dynamo at 50 volts, connected directly to and driven by a standard 10 H.-P. Westinghouse engine, having 5½-in. steam cylinders and a 5-in. stroke.

Elevator Plant.—There are 15 freight and baggage elevators and six passenger elevators, all operated by hydraulic power and of the Stokes & Parrish type. The freight and baggage elevators are worked by rams under a pressure of 250 lbs. per square inch at a speed of 80 ft. per minute at half load. Three pumps, two accumulators and a return tank are located in the Cherry Street power-house of sufficient capacity to operate the entire plant, and two pumps, one accumulator and a return tank are placed in the engine-room of the head-house. These separate installations are arranged to operate either together or separately, one being the relay of the other. During the winter months the head-house pumps may be run in order to allow the exhaust steam to be used for heating purposes. The pressure and return mains serv-

ing the various elevators are placed in the tunnels and ducts under the exterior corridors of the cold storage cellar, and are controlled by valves, so that either or both of the operating plants may be used. In case of an accident to the main pipes the damaged portion may be cut out without impairing the efficiency of the service.

Each accumulator has a piston of 17 ins. diameter, with a stroke of 6 ft., and is loaded with 32 tons. The pumps are of the Worthington compound duplex pattern with outside-packed pistons, having 14-in. high-pressure and 20-in. low-pressure cylinders, $7\frac{3}{4}$-in. water plungers and a 15-in. stroke.

The return tank in the Cherry Street power-house has a capacity of 650 cu. ft., that in the head-house 340 cu. ft., and the force and return mains are 7 ins. in diameter.

The table on page 156 gives the general data in reference to the various freight and baggage lifts.

Of the six passenger elevators, four of them are rope hoist and two of them operated by rams, working under a pressure of 110 lbs. per square inch. The apparatus, which is hydraulic and pneumatic, its location being shown on Plate I, is as follows: Two Worthington duplex compound pumps with 14-in. high-pressure and 20-in. low-pressure cylinders, 12-in. water plungers and 10-in. stroke. Two horizontal wrought-iron pressure tanks each 22 ft. long and 5 ft. in diameter. One auxiliary vertical pressure tank 4 ft. in diameter and 6 ft. long. One return water tank having a capacity of 660 cu. ft., and one auxiliary tank of 66 cu. ft. One hydraulic engine with 18-ft. 1-in. stroke, $22\frac{1}{2}$-in. cylinder, geared 8 to 1, two steel $1\frac{5}{16}$-in. piston rods. One hydraulic engine with 17-ft. stroke, $22\frac{1}{2}$-in. cylinder, geared 8 to 1, two $1\frac{5}{16}$-in. piston rods. One hydraulic engine with 20-ft. 5-in. stroke, 18-in. cylinder, geared 6 to 1, two $1\frac{5}{16}$-in. piston rods. One hydraulic engine with 20-ft. 5-in. stroke, 18-in. cylinder, geared 6 to 1, two $2\frac{3}{16}$-in. piston rods.

The auxiliary pressure and return tanks are used to equalize the flow of water to the pumps from the elevators in the west end of the building. The water is pumped from the return tank to the pressure tanks (which are kept half full of air by means of snifting valves) and is delivered from them to the cylinders of the hydraulic engines. One duplex pump and pressure tank is sufficient to operate the elevators, and the duplicates are held in reserve.

TABLE GIVING GENERAL DIMENSIONS AND DATA OF BAGGAGE AND FREIGHT ELEVATORS FOR THE PHILADELPHIA AND READING TERMINAL RAILROAD STATION, PHILADELPHIA, PA.

LOCATION.	Style.	Lift.	Size of car.	Net capacity in pounds.	Speed with one-half load per minute.	Working pressure on piston not to exceed	Diameter of ram or piston.	REMARKS.
South end of train-shed { Outgoing baggage	Direct ram	23′ 3″	8′ 0″ x 13′ 6″	6 000	80′	300 lbs. per square inch.	8½″	Lift from street to track level.
South end of train-shed { " "	"	23′ 3″	8′ 0″ x 13′ 6″	6 000	80′	300 " "	8½″	" " "
South end of train-shed { Incoming baggage	"	23′ 3″	8′ 0″ x 13′ 6″	6 000	80′	300 " "	8½″	" " "
South end of train-shed { " "	"	23′ 3″	8′ 0″ x 13′ 6″	6 000	80′	300 " "	8½″	" " "
Filbert Street	"	11′ 9″	7′ 0″ x 3′ 6″	2 240	80′	300 " "	5½″	" basement to street.
" "	"	12′ 0″	7′ 0″ x 3′ 6″	2 240	80′	300 " "	5½″	" " "
Twelfth Street	"	35′ 8″	7′ 6″ x 13′ 3″	6 000	80′	300 " "	8½″	" " track level.
" "	"	35′ 8″	7′ 6″ x 13′ 3″	6 000	80′	300 " "	8½″	" " "
" "	"	35′ 8″	7′ 6″ x 13′ 3″	6 000	80′	300 " "	8½″	" " "
" "	"	35′ 8″	7′ 6″ x 13′ 3″	6 000	80′	300 " "	8½″	" " "
Arch Street	"	12′ 6″	8′ 6″ x 12′ 2″	6 000	80′	300 " "	8½″	" " street level.
East of shed, etc.	"	35′ 8″	7′ 6″ x 13′ 3″	6 000	80′	300 " "	8½″	" " track level.
" "	"	35′ 8″	7′ 6″ x 13′ 3″	6 000	80′	300 " "	8½″	" " "
" "	"	35′ 8″	7′ 6″ x 13′ 3″	6 000	80′	300 " "	8½″	" " "
" "	"	35′ 8″	7′ 6″ x 13′ 3″	6 000	80′	300 " "	8½″	" " "

One of the elevators is arranged to lift 6 000 lbs., so that safes and other heavy articles may be conveyed by it. The ram elevators are served from the pressure tanks. The elevators are provided with safety apparatus and speed governors which arrest the descent of the car if it exceed a safe speed from any cause. Should this apparatus fail, the car drops into an air cushion 17½ ft. in depth, which is ample to prevent breakage of the car or loss of life.

The table on page 158 gives the general dimensions of the passenger elevators.

Water Supply.—The water supply is from the city mains and from the rainfall on the roof of the head-house. In the basement of the head-house there are two Jewell pressure filters, one filtered water tank and three Worthington pumps; in the attic there are eight storage tanks, which have been previously mentioned in the general description of the building, and in the Cherry Street power-house there are two Worthington pumps, a brick cistern and a pressure tank (see Plates I and II).

Water from the city mains on Market Street is supplied to the filters through a meter, and a direct supply, without a meter, with sealed valve, is also furnished for fire service. The water through the filters serves the head-house and the pipes delivering at the end of each track in the train-shed for a supply to passenger cars; the rainwater collected from the roof, supplemented from the pumps when necessary, is used to flush the water-closets; and the market, restaurant, etc., under the train-shed, are furnished directly from the water mains on Twelfth Street. The power-house is served by a 6-in. main connected with the city mains on Arch and Cherry Streets, and fire hydrants in the cold storage cellar and in the train-shed are furnished from the Arch Street main as well as from the pumps serving the ram elevator system, check valves being introduced so that a pressure of 250 lbs. per square inch can be brought into service if required. The cistern and pump in the power-house are held in reserve in case of accident to the city mains or in the event of the city pressure falling below the pressure that is required to serve the cold storage machinery.

The two filters in the head-house have a total capacity of 20 000 galls. per hour, each consisting of a wrought-iron shell 8 ft. in diameter, containing 200 discharge nozzles ⅜ in. in diameter fitted with finely perforated rose screens of aluminum bronze 1¾ ins. in diameter.

TABLE GIVING GENERAL DIMENSIONS AND DATA OF PASSENGER ELEVATORS FOR THE PHILADELPHIA AND READING TERMINAL RAILROAD STATION, PHILADELPHIA, PA.

LOCATION.	Style.	Lift.	Size of car.	Net capacity in pounds.	Speed with one-half load per minute.	Working pressure on piston not to exceed	Diameter of ram or piston.	REMARKS.
Head-house, west...	Rope hoist.	133′	6′ 0″ x 5′ 6″	2 240	500′	110 lbs. per square inch.	18″	Lift from cellar to sixth story.
" " . .	"	123′	6′ 0″ x 5′ 6″	2 240	500′	110 " " "	18″	" basement to sixth story.
" east,...	"	123′	6′ 9″ x 4′ 9″	2 240	500′	110 " " "	18″	" " " "
" " ,...	"	123′	6′ 9″ x 4′ 9″	*6 000	500′	110 " " "	18″	" " " "
Waiting-room	Direct ram.	23′ 3″	6′ 0″ x 8″	3 000	200′	110 " " "	9″	" street to track level.
"	"	23′ 3″	6′ 0″ x 8″	3 000	200′	110 " " "	9″	" " " "

* 6 000-lb. load is for lifting safes at slow speed. Passenger load to be 2 240 lbs.

The water from these nozzles passes through a bed of crushed quartz 2½ ft. in depth. In cleaning the filters the direction of the water is reversed and is admitted to the underside of the quartz bed, which is agitated at the same time by a revolving comb operated by an electric motor. For the purpose of sterilizing the filters, high-pressure steam can be admitted to the discharge chamber and forced back through the nozzles. The tank for filtered water, also in the basement of the head-house, is built of wrought iron in two compartments and is of 10 000 galls. capacity.

The three pumps in the same location have the following dimensions : Two Worthington compound duplex pumps, 10-in. high-pressure and 16-inch low-pressure cylinders, 8-in. water plunger and 10-in. stroke ; and one Worthington fire pump, 17-in. steam cylinder, 10-in. water plunger and 15-in. stroke, the pumps being connected so that they can be operated either separately or together.

The tanks in the attic are of wrought iron and have a capacity of 27 600 galls. Fire nozzles are provided in the various stories of the building at convenient points and are supplied directly from the tanks, as well as from the fire main, being so controlled by check valves that when the fire main is in service the high pressure from the pumps can be utilized automatically. The cistern in the power-house is of brick, laid in cement, and is 9 ft. in diameter and 13 ft. deep. It is fed by a supply from the city mains. Connected with this cistern are two Worthington duplex pumps, having 7½-in. steam cylinders, 7-in. water plungers and a 10-in. stroke, and a pressure tank 3 ft. in diameter and 12 ft. long.

Drainage.—Drainage wells are provided for all wastes draining below the levels of the city sewers, and steam pumps to discharge their contents into the sewers.

Cold Storage.—It has already been stated that the basement or cellar under the market-house is occupied by cold storage rooms. The space between the exterior walls and the first row of pier columns, all around the basement, is reserved as a wide passageway, and under its floor is a duct to carry service pipes from the tunnel to elevators, etc., as previously described. The whole of the area inside of the outer rows of pier columns is intended to be ultimately filled with cold storage rooms and corridors, and the air-locks in connection with them. At present about two-thirds of the space is so occupied. The

total capacity of the various rooms, when complete, will be 377 940 cu. ft., and of the corridors 63 450 cu. ft. There are already constructed 236 460 cu. ft. of rooms and 43 740 cu. ft. of corridors. The rooms are of different sizes, varying from about 5 000 to 17 000 cu. ft. each, and have a clear height under the drip-boards, near the ceiling, of 6 ft. 9 ins. They are arranged to be used at different temperatures varying from 3° to 44° Fahr., and were constructed for a test temperature of 5° above zero as a maximum which had to be maintained throughout the entire system of empty rooms for at least 36 continuous hours in the months of June, July or August. Particular care must be taken in the construction of such rooms, to arrange that, when in use, the air in them shall be kept very dry. This is satisfactorily accomplished by means of air-locks having doors closing automatically. The freezing-rooms are each provided with five such air-locks and the other rooms with three or four, as may be necessary (see Plate I).

The lumber used in the construction of these rooms is dressed spruce, a material that does not communicate any taste to articles placed in storage. The outside partitions and ceilings have six thicknesses of boards with two air spaces of 1 in. each, and the inside and cross-partitions have generally four thicknesses of boards, with one air space of 4 ins., except for freezing rooms, where they are like the exterior walls. Partitions for swinging doors in corridors and vestibules have two thicknesses of boards, one vertical and one horizontal, without any air space, nailed to 2 x 4 in. studs and inter-ties. The boards in all cases are thus arranged in pairs, and between the boards of each pair there are three layers of red Neponset paper, 1 oz. per square foot, with 3-in. lapped joints. All air spaces are divided by strips forming cells to prevent circulation of air, and in the ceiling to form nailing pieces for boarding.

The floor is of asphalt, $\frac{3}{4}$ in. thick, laid on a concrete base. Cement curbs are formed for the partitions and doorways, overlaid with asphalt, made airtight and protected at the doorways by wrought iron plate coverings set in the asphalt. All sills for partitions are bedded in asphalt. Secondary floors of spruce are laid in the freezing-rooms over the asphalt with an air space between.

The doors to cold storage rooms are built over an inside frame of stiles and rails $1\frac{3}{4}$ ins. thick, paneled flush with narrow matched $\frac{3}{8}$-in. boards rabbeted in and having three layers of red Neponset

paper underneath, the joint between boards and frame being calked tight. Three layers of the same paper are placed on both sides of the doors, and secured by canvas covering over felt edging; outside stiles and rails are then put on and diagonal boarding cut in between.

The corridor doors are made of $1\frac{3}{8}$-in. matched boards, planed and beaded, with all outside edges rounded, secured together by $\frac{7}{8}$-in. ash battens, placed on both sides and having galvanized iron clinch nails passing through both battens. Particular care has been taken in the construction of the frames for the doors and in hanging them to prevent the admission of air around them. The edges of the doors are beveled and the felt on them secures tight, close fitting when the doors are shut. All doors are double, and air-lock doors are double swing.

Overhead wrought-iron tracks for handling and transporting material for storage are provided throughout certain of the rooms and corridors. They are carried by hangers, made perfectly level and furnished with swinging bars, hinged at one end, as switches, so that change may be made to any track desired. No curve is used of less radius than 7 ins., and the rollers for the tracks are made as large as practicable, the wheels being of cast-iron and the frames and hooks of wrought iron.

Drip-boards are formed of two $\frac{3}{8}$-in. thicknesses of spruce, each planed and matched, laid in opposite directions and having three sheets of Neponset paper between them. On these are $\frac{3}{8}$-in. battens, both crosswise and lengthwise, with three additional layers of Neponset paper on top. A second set of crosswise battens is then placed over the first, with corrugations cut in them; and on these galvanized corrugated iron is laid, another batten being placed at the upper end, also cut to suit the corrugations, to keep the iron in place. Angle irons are used underneath for additional stiffening. The boards are fastened together with galvanized iron clinch nails, and the corrugated iron is secured by rivets. The drip-boards are hinged at the lower end and secured by hooks at the upper end. They have a fall of about 1 in 60. Gutters are placed at the lower ends of the drip-boards, formed of spruce with white lead joints. They connect with gutters at the ends of the rooms, and from these 2-in. wrought-iron water pipes pass through the partitions to the corridors or vestibules, provided with bends and check valves on the outside where the drip can be collected in buckets. Every care is taken to calk all openings and make them thoroughly air-

tight; all those for track hangers are protected from leakage; all drip-boards abutting against columns have curbs to prevent water from running down the column, and there are innumerable details that cannot be stated in this paper.

The temperatures in the various rooms are maintained within 2° of that required for the various conditions, which, as already stated, vary from 3 to 44°. Brine is used for cooling the rooms and is circulated in coils of 1½ ins. diameter through wrought-iron pipes, there being at least four independent coils in each room; they are usually placed overhead above the drip-boards, the latter aiding to induce a circulation of the air. The coils are so arranged that the entire content of the room is equally cooled with one or more coils in service, and generally there is an area of cooling surface equal to the square feet of ceiling in the room, an increase beyond this being made in the freezing-rooms to meet requirements. The refrigerating apparatus consists of ammonia compressors, ammonia condensers, brine tanks, brine pumps and mains. The apparatus, except the mains, is in duplicate.

The condensing pressure for the ammonia is 145 lbs. per square inch and the evaporating pressure for the ammonia is 5 lbs. per square inch. The difference of temperature between initial and waste water in the condensers is 10° Fahr., and the difference of temperature between the initial and return brine is 3°.

There are two vertical Boyle ammonia compressors specially designed for this plant, each having a capacity for refrigeration equal to 75 tons of melting ice in 24 hours. Each compressor has two single-acting ammonia cylinders of 15 in. diameter and 30-in. stroke, driven by a Corliss engine, with 20-in. cylinders, 36-in. stroke, and two fly-wheels of 8 ft. diameter. The shaft of the engine is 10 ins. in diameter, and the speed 60 revolutions per minute, giving 83 rated H. P. with 80 lbs. steam pressure, and one-fourth cut off.

There are three independent submerged wrought-iron condenser tanks, each containing 7 865 lin. ft. of 1-in. pipe, and three wrought-iron brine tanks, insulated with ground cork and finished with pine, having a total capacity of 67 700 galls. and containing 35 788 lin. ft. of 1-in. pipe. The pipe coils in the condensers and brine tanks are arranged in sections with valves, so that any coil may be removed and cleaned or repaired without placing the tank out of commission. The

brine is pumped to the cold storage rooms by two Snow duplex steam pumps, having 14-in. steam cylinders, 12-in. water plungers and 14-in stroke.

Insulation.—All high-pressure steam pipes (that is, carrying a pressure of 100 lbs. above the atmosphere) are covered with magnesia sectional covering 2 ins. thick, and all low-pressure and exhaust pipes with the same kind of covering 1 in. in thickness. The brine mains have the patent covering of Macan & Company, of Philadelphia, consisting of 3 ins. of hair felt, in two layers of 1½ ins. each, breaking joint, each layer being coated with a preparation of gutta-percha, and the whole covered with waterproof paper and painted canvas. The insulation has proved very satisfactory. The two 6-in. steam mains from the boilers to the head-house, although they run a distance of 1 200 ft., of which 185 ft. is in the tunnel carrying at the same time two 8-in. brine mains, show no appreciable difference in pressure in the head-house, and the difference in temperature of the brine is but 3° between the the initial and return. No ice or moisture forms on the brine pipe covering throughout its entire length.

As a matter of interest, it may be stated that the average daily consumption of coal for the entire plant of the station from January 1st, 1894, to May 1st, 1895, was as follows:

1894.	Tons.	1895.	Tons.
January	30.3	January	46
February	37.1	February	52
March	32.4	March	42
April	28.7	April	35.8
May	33.7		
June	26.6		
July	32		
August	40		
September	30		
October	38		
November	40		
December	45		

It is to be noted that the large increase of coal consumption in August, 1894, is partially due to dirty boilers and partially to the fact

that steam was supplied for an ice plant placed in the power-house to furnish ice for use in passenger cars, etc. The various tests of boiler evaporation gave the following results, the first two having already been noted under boilers:

First.—February 25th, 1893, clean boilers, fires forced, 9.74 lbs. water from and at 212° Fahr. per pound of combustible.*

Second.—April 29th, 1893, clean boilers, fires not forced, 9.90 lbs. water from and at 212° Fahr. per pound of combustible.*

Third.—July 18th, 1894, dirty boilers, fires not forced, 8.62 lbs. water from and at 212° Fahr. per pound of combustible.†

Fourth.—July 19th, 1894, dirty boilers, fires not forced, 9.60 lbs. water from and at 212° Fahr. per pound of combustible.†

Fifth.—July 25th, 1894, dirty boilers, fires not forced, 9.38 lbs. water from and at 212° Fahr. per pound of combustible.‡

Plate IV, Fig. 1, is interesting as showing the work of construction of the building in progress. The end at Arch Street was being painted while the foundations towards Market Street were still being dug out, and the train-shed roof only partially erected. At the same time, the market under the train-shed floor was occupied and in use. Plate IV, Fig. 2, is an exterior view of the station building.

Cost of the Buildings.—As a general rule it is rather a delicate matter with the engineer or architect to make mention of the cost of the work under his charge, as it involves questions in confidence between himself and his client. The subject, however, is one of special interest to the professional man, and it is felt that no confidence is violated in giving the following general information.

The cost of the head-house proper, exclusive of heating, electric lighting, elevators, water supply, pumping, filters and other appurtenances to these special items, was $35.95 per square foot of superficial area, or 23.1 cents per cubic foot of content, computed between outside faces of exterior walls and from cellar floor to top surface of roof. The train-shed and lobby, including the foundations, the cold storage cellar and rooms, the tunnels, the market house with stalls and all fixtures, and the restaurant, but excluding brine pipes in cold storage rooms, electric lighting, heating, elevators and water supply, was $8.53 per

* Red ash pea coal, Lykens Valley.

† White ash buckwheat coal, St. Clair mine.

‡ White ash pea coal, St. Clair mine.

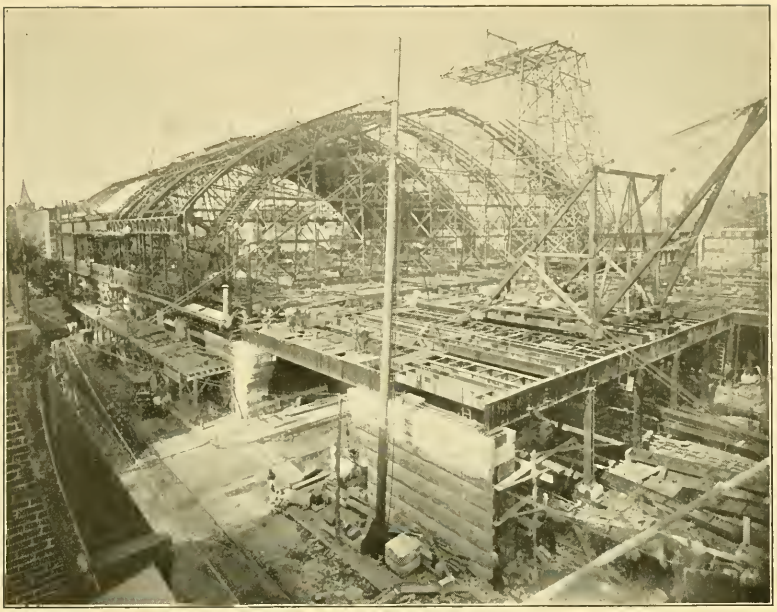


FIG. 1.

FIG. 2.

square foot of surface covered. The structural iron work of the roof to the train-shed cost $1 15 per square foot of surface covered.

There were 885 scale and 225 full-size drawings made for the buildings, including the Spring Garden Street station and the engine houses, as well as 500 drawings for the railroad work, a total of 1 610 drawings. There were over 7 000 blue prints made from these drawings for use on the work.

In the execution of this work by the firm of Wilson Brothers & Company as engineers and architects, John A. Wilson, M. Am. Soc. C. E., acted as chief engineer for the entire operation, taking particular charge of the railroad work, and having William H. Millard as principal assistant. He also superintended all of the construction, including the buildings as well as the railroad lines. Joseph M. Wilson, M. Am. Soc. C. E., and Henry W. Wilson, M. Am. Soc. C. E., together with Howard S. Richards as principal assistant, designed and had charge of the bridges, roof and all structural work. Henry A. Macomb and Joseph M. Wilson, with John J. Dull as principal assistant, designed and had charge of the architectural work. Charles G. Darrach, M. Am. Soc. C. E., with Robert C. Clarkson, Jun. Am. Soc. C. E., as principal assistant, designed and had charge of all the mechanical engineering work, including the electric lighting and refrigerating plants, and all other machinery.

APPENDIX I.

SPECIFICATIONS FOR THE EXCAVATION AND MASONRY ON THE PHILADELPHIA AND READING TERMINAL RAILROAD COMPANY'S LINES IN THE CITY OF PHILADELPHIA.

The structure of the Philadelphia and Reading Terminal Railroad (excepting where constructed of iron) will generally consist of two retaining walls parallel with the center line of the road, and abutment walls at the street crossings, built on the street lines. The area enclosed by these walls will be filled hereafter with material suitable for embankment. In special cases piers of masonry will be built to carry the iron superstructure. These walls will be built in the positions and of the heights shown by the maps and profiles, and of the thickness, form and dimensions shown by the drawings to be hereafter prepared to suit the various places. The profiles and drawings exhibited at the letting of the work are general ones, and will be modified by the engineers as occasion may require.

In addition to the masonry for the structure proper, there will be foundation piers for iron columns, masonry for street bridges over the railroad, etc., all of which are intended to be covered by this specification. In the case of foundations for columns, anchor bolts are to be built into the masonry, and cap-stones are to be provided as indicated by the detailed drawings.

There will be two distinct classes of masonry, viz.: first-class rock range work, which will generally be used for abutments, piers and foundations of columns; rubble work, which will generally be used for retaining walls.

The engineer will designate what class of masonry is to be used at each place, and the right is reserved to use either first-class masonry or rubble work in any position at the discretion of the engineer.

All materials used throughout must be the best of their respective kinds, and the entire work is to be constructed and finished in every part in a good, substantial and workmanlike manner, according to the drawings and these specifications, to the full extent and meaning of the same, and to the entire satisfaction, approval and acceptance of the engineer, and under the supervision and directions of such agent as he may appoint.

Alterations.—It is understood that the company shall have the right to make any alterations, additions or omissions of work or materials herein specified or shown on the drawings, during the progress of the work, that they may find to be necessary, and the same shall be acceded to by the contractor or contractors, and carried into effect without in any way violating or vitiating the contracts.

Also, that any disagreement or difference between the company and contractor, upon any matter or thing arising from these specifications or the drawings to which they refer, or the kind or quality of the work

required thereby, shall be decided by the engineer, whose decision and interpretation of the same shall be considered final, conclusive and binding upon both parties.

Care of Finished Work.—Particular care must be taken by the contractor of all the finished work as the walls progress, which work must be thoroughly protected from injury or defacement during the erection and completion of the walls.

Removal of Rubbish, Etc.—All refuse material and rubbish that may accumulate during the progress of the work is to be removed from time to time, as may be directed by the agent in charge; and on the completion of the work, the streets and grounds must be thoroughly cleaned up, and the surplus material and rubbish removed.

Risks, Blame, Etc.—The contractor is to assume all risks and bear any loss occasioned by neglect or accident during the progress of the work, until the same shall have been completed and accepted by the engineer. He is also to assume all blame or loss by reason of neglect or violation of state, city or district ordinances, laws or regulations, encroachments upon neighbors, or from any other cause.

The engineer shall have full power, at any time during the progress of the work, to reject any materials he may deem unsuitable for the purposes for which they are intended, or which are not in strict conformity with the spirit of these specifications. He shall also have the power to cause any inferior or unsafe work to be taken down and altered at the cost of the contractor.

Excavation of Foundations.—The excavation of foundation trenches shall be done as the work requires, and the materials taken from the trenches, excepting that needed for refilling alongside of walls, shall be deposited at such places, in the area enclosed by the walls, or elsewhere, as the engineer may direct from time to time.

The foundation trenches will be excavated at least 6 inches wider on each side than the width of the masonry, and to such depth as may be directed by the engineer. Generally, the sides of the foundation trenches will be excavated vertically, and the contractor must provide such shoring as may be necessary to prevent caving of the sides of the trenches until the masonry is built. In special cases where, in the judgment of the engineer in charge, the nature of the material or the special circumstances existing require that the sides of the trenches shall be sloped, they will be excavated in that way to the stakes given by the engineer. In all cases the width, depth and side slopes of the trenches will be regulated by the engineer, and no material excavated outside of the lines given by him will be paid for.

All old wells and privies found within the line of the road-bed must be thoroughly cleaned out and filled up with good material, and where such wells occur under walls, they must be also properly arched over.

When the foundation masonry has been built, the spaces between the walls and the sides of the excavation must be carefully and thoroughly refilled to the height of the adjacent surface.

All excavation of foundation trenches will be estimated and paid for by the cubic yard, measured in the excavation, which price will include refilling the trenches alongside the walls, the furnishing of timber for shoring, pumping water, the depositing and spreading of the excavated material, as may be directed by the engineer in charge, and all incidental work and expenses connected therewith.

Foundations.—Where good, solid, natural foundations can be had, the masonry will be built on the same. If, in the judgment of the

engineer, the character of the foundation requires it, concrete shall be used to prepare a foundation for the stone work. Such concrete shall be formed of an approved, hard, durable stone, broken into angular fragments of a size to pass through a 2½-in. ring, and to be screened. The concrete shall consist of broken stone, 6 parts; clean, sharp, coarse sand, 2 parts; and first quality Portland cement, 1 part.

The cement and sand are first to be spread in layers and thoroughly mixed dry in a box or on a platform provided for the purpose. This mixture is then to be removed to a box from time to time as required, and made into mortar, sufficient water being added to make a stiff paste, after which the broken stone is to be added with sufficient water to bring the mass to a proper consistency, the materials being manipulated as the water is added, until all are thoroughly incorporated, and each piece of stone is well coated with the mortar, and the broken stone distributed equally through the entire mass. This concrete must be placed in the foundation trench in layers not over a foot thick, filling the whole width of the trench. Each layer must be well rammed until a film of water appears on the surface, but not enough to make it quake.

The concrete must not be mixed in larger amounts than requires 1 barrel of cement at a time, and must be used as mixed, not being allowed to lie and then remixed with water. No concrete that has commenced to set will be allowed.

Masonry must not be laid on concrete until it has set firmly and becomes hard. Should special conditions require that foundations be otherwise prepared, the work shall be done as directed by the engineers.

Rubble Masonry.—All the masonry of retaining walls shall be rubble work, laid with cement mortar in irregular courses, and will consist of stones containing generally 12 cu. ft. each, so disposed as to make firm and compact work, and no stone in the work shall contain less than 4 cu. ft., except for filling up the interstices between the large blocks in the heart of the wall; provided, however, that in the upper part of the wall, where its thickness becomes less than 5 ft., the average size of the stones shall be from 6 to 8 cu. ft. to suit the thickness of wall. At least one-fifth of each face of the wall shall be composed of headers extending full size not less than 4 ft. into the wall (excepting where the wall is less than 5 ft. thick, when the headers shall extend across two-thirds of the width of the wall) so arranged that a header in one face shall be between two headers in the opposite face. When the headers from the two faces do not interlock, stones not less than 4 ft. long shall be laid transversely in the heart of the wall to connect the two opposite sides. If, in the progress of the work, an increase in the number of headers specified should be required by the engineer such additional number as he shall designate shall be laid in the work. The corner-stones shall be neatly hammer-dressed so as to have horizontal beds and vertical joints.

Where the retaining walls join the abutment walls, the stones must be squared and jointed to ensure close contact, but the retaining and abutment walls will not be bonded together. The masonry must be accurately built to the lines and levels given by the engineer, and the exposed faces must be straight, true and smooth, without prominent or sharp projections of the rock or quarry face of the stone.

The stone used must be first-class Conshohocken stone, or stone of equal quality, to be approved by the engineer. It must be hard and durable, of good size and shape, with good natural beds suitable for

making first-class rubble work; no stone in the work to have less bed than face, and all must be carefully bedded on their broadest faces, and well bonded together.

The interior and exterior faces of the wall above ground shall have the joints thoroughly cleaned out for not less than 1 in. in depth, and they shall be pointed with Portland cement mortar.

Retaining walls are to be finished at top with coping as per drawings. The coping stones to be neatly dressed, carefully bedded in Portland cement mortar, with grouted joints, secured to wall by dowels, and to each other by clamps.

Drainage openings are to be made in walls, or pipes built in, as may be directed by the engineer.

First-Class Masonry.—Abutment walls and piers shall be first-class, rock-range masonry, of large-dimension stones laid in cement mortar and carefully grouted. The stone used to be Conshohocken stone, or stone of equal quality, to be approved of by the engineer. The stone to be accurately squared, jointed and bedded over their whole surface, and laid in courses not less than 12 ins. thick, nor exceeding 24 ins. thick, regularly decreasing in thickness from bottom to top of abutment. The stretchers shall in no case have less than 16 ins. bed, nor less bed than face, and shall generally be at least 4 ft. long. The headers shall be of similar size with the stretchers, and shall hold the size in the heart of the wall that they show on the face, and shall occupy at least one-fifth of the face of the wall. When the wall is too thick to admit of the headers interlocking, stones not less than 4 ft. long shall be placed transversely in the heart of the wall to connect the opposite sides. The stones for the heart of the wall must be of the same thickness as those in the face, bedded the same as the face stone, but not jointed, and must be well fitted to their places, the interstices being filled with sound stones. The abutments must be finished with neatly cut coping, as per drawings, and the whole work neatly pointed with cement mortar.

N. B.—The foundation masonry below ground will be of same quality and character as the masonry above ground.

Mortar.—The mortar used for all classes of masonry shall be fresh mixed, and composed of 1 part by measure of Portland cement of approved quality and 2 parts of clean, sharp sand. The cement and sand shall be thoroughly mixed together dry, and a portion of the mixture be worked up with water for immediate use, as may be needed from time to time. No cement mortar shall be used that has been mixed with water more than 10 minutes, and no cement mortar shall be worked over a second time.

The sand used must be sharp, clean and entirely free from mica, loam, clay or other impurities.

All cement used for mortar or grouting shall be first quality American, English or German Portland, and shall be subject to the tests described in the Standard Specification of Wilson Brothers & Co. for cements, hereunto attached, and which are hereby made a part of this specification.

General Conditions.—Particular care must be taken in selecting the stone for the exposed faces of the walls above ground. Masonry must not be laid in hard-freezing weather, and work left standing over winter must be covered up and protected. All masonry will be paid for by the cubic yard of 27 cu. ft., engineer's measurement, of actual cubic content, without regard to any measurers or trade rules or customs of measurement, and all measurements and calculations of quantities will

be made by the engineer in charge of the work. The price paid per cubic yard shall in every case include the furnishing of all materials and labor, all the expense of delivering the materials, and the cost of all scaffolding, shoring, derricks, tools, etc., and all expenses and risks of every kind.

Especial care must be taken, and every precaution used during the progress of the work, to protect the streets and to avoid injury or inconvenience to persons passing. Where the abutment or other walls front on streets, on the completion of each wall the sidewalk must be restored and repaved, and curbs reset (if they have been disturbed) and be left in good condition. Where any of the walls come close to adjoining properties, the necessary precautions must be taken to ensure the safety of adjoining buildings, and the foundation walls of such building must be underpinned if necessary to protect them. The contractor must assume all responsibility for injury to persons or property. When directed by the engineer, temporary openings must be arranged for in the walls to admit of the taking in of filling material for the embankment.

No charge shall be made by the contractor for hindrances or delay, from any cause, in the progress of any portion of the work in this contract, but it may entitle him to an extension of time, allowed for completing the work, sufficient to compensate for the detention, to be determined by the Chief Engineer, provided he shall give the engineer in charge immediate notice, in writing, of the cause of the detention.

Nor shall any claim be allowed for extra work, unless the same shall be done in pursuance of a written order from the engineer in charge, and the claim made at the first settlement after the work was executed, unless the Chief Engineer, at his discretion, should direct the claim, or such part as he may deem just and equitable to be allowed.

All of the work to be done in the best and most workmanlike manner, of approved materials, according to the plans and drawings, and everything necessary for the proper and complete execution of the said plans and drawings, whether the same may have been herein particularly specified or not, or indicated on the plans referred to, to be done and furnished in a manner corresponding with the rest of the work, as well, as truly and as faithfully as though the same were herein particularly described and specifically provided for.

Every part of the work to be executed under the direction and subject to the approval of the engineer, to whom all questions relating to the intent and meaning of the plans, drawings, specifications, or of the kind or quality of the materials and work required thereby, shall be referred, and his decision shall be final, conclusive, and without appeal.

Excavation of foundations will be paid for by the cubic yard.

All masonry, including coping, will be paid for by the cubic yard, the coping being measured in as part of the wall to which it belongs.

Concrete in foundations will be paid for by the cubic yard.

Special work that cannot be determined in advance, such as cleaning and arching privies and wells, underpinning foundations of adjacent buildings, etc., will be paid for at engineer's estimate, viz., cost, with 10% added for superintendence and use of tools.

Philadelphia, February 24th, 1891.

SUPPLEMENTAL SPECIFICATIONS FOR ARCH BRIDGES OVER STREETS BETWEEN CALLOWHILL AND ARCH STREETS, ON THE LINE OF THE PHILADELPHIA AND READING TERMINAL RAILROAD.

Arch bridges are to be built according to the detailed plans furnished and under the specifications for the Reading Terminal masonry, dated February 24th, 1891, so far as the same will apply, the same being supplemented as hereinafter stated.

Arches proper to be of brick, as per drawings. The brick to be the best quality of straight, hard-burned, hand-made brick. No bats, salmon or soft brick of any grade will be allowed. The brickwork will be laid in cement mortar, with joints not exceeding ¼ in., and with such bond as the engineer may direct. The bricks at the time of laying to be well wet, the cement mortar to be thin and freshly mixed, the cross-joints to be carefully filled, and each course to be thoroughly grouted. The several courses of brick at the crown, forming the key, to be laid close, with very thin joints, and thoroughly grouted.

The face of the arch will be formed of ring stones composed of alternate long and short bond stones, not less than 3 ft. and 18 ins. respectively, neatly cut to a pattern. Ring stones to be of Leiperville, Conshohocken or Port Deposit stone, as directed. The exposed face on the facade to be quarry face with a draft line all round each stone. Parapet or facade masonry to be first-class rock range work, as specified in the general specifications for piers and abutments. Similar masonry to be used on the street fronts of abutments, with the stones forming the continuation of the arch in the interior of the walls, to be arranged as shown on drawings, all neatly and accurately cut, coping to be as per drawings.

The foundations below the first-class work will be concrete and rubble work as per drawings. The backing over the arch and portions of abutments above the arch line to be rubble work as per drawings, the stones where they join the brickwork to be laid in close to the arch. All the rubble work to be first class, closely laid, of large stones, and thoroughly grouted. The whole work to be laid in mortar composed of 2 parts of clean sharp sand and 1 part of best American Portland cement, of approved quality, except brickwork, which shall be laid in mortar of best imported Portland cement. Bar sand to be used for mortar in brickwork. Grout to be of same cement and sand, and with same proportions.

The whole work to be finished in a good and workmanlike manner, so as to be impervious to water. All exposed faces of brick and stone work to be neatly pointed with Portland cement mortar. Cast-iron drain pipes are to be built in the walls as directed, and the upper surface of masonry to be covered with two layers of Neuchatel asphalt covering, each layer 1 in. thick.

The price paid for the masonry per yard will be for all complete, from the bottom of the excavation (including concrete), and will cover centering and all incidental expenses, except the drainage pipes and asphalt covering, which shall be paid for at engineer's estimate.

PHILADELPHIA, August 3d, 1891.

APPENDIX II.

PHILADELPHIA AND READING TERMINAL RAILROAD.

The iron construction on this road will consist of the following structures :

First.—Plate-girder deck bridges over streets from Arch to Callowhill, for four or more tracks, as shown by the detailed plans of the road.

Second.—On the branch from Callowhill near Twelfth Street, to Broad and Noble Streets, deck plate-girder construction for two tracks, as shown by general drawings, from Callowhill Street to a point west of Twelfth Street ; iron construction of columns and girders to Thirteenth Street, and a plate-girder with floor on lower chords over Thirteenth Street.

Third.—From near Eleventh and Callowhill Streets, to a point on Ninth Street north of Green, iron construction of columns and plate-girders for two tracks, with a through truss bridge over Spring Garden Street, and plate-girder with track on lower chords over Green Street; all as shown on the general drawings.

The several structures must be designed to suit the particular localities which they are to occupy, and for the number of tracks required at each place, and must include sidewalks on each side of the structure for trainmen. All bridges and portions of iron construction over streets must be provided with acceptable screens, arranged as per general drawings, to guard the view from the streets of passing trains, and approved system of deadening for floors must be provided for the full width between house lines, over all streets. All plans must conform to the requirements of the ordinance of City Councils permitting the construction of the road, and are subject to the approval of the Department of Public Works of the City of Philadelphia. The profile of the road will show the elevations of track and the clearances required at each street. General drawings and specifications will be furnished on application to the chief engineer.

The contractor for the iron work must furnish anchor bolts and attachments for building into the masonry of column foundations, with drawings for locating same. He must also test for himself, on the ground, all measurements of spans of bridges, and distances between column foundations, construct the iron work to conform to same, and be responsible for any errors of measurements or variations that may be found in the iron work when same comes to be erected.

Where the line of road is on a curve, the proper elevation of the outer rail (to be established by the engineer) is to be arranged and provided for, the elevation being tapered off on the tangent as may be directed by the engineer.

The bridges and iron structure are to be finished complete in every respect, including the wooden floor system, sidewalks and railings, ready for laying the rails of the track. All iron work to be painted with three coats of best white lead in linseed oil to tints as may be

directed by the engineer. It must be understood that this painting is to be in addition to the priming coat of paint provided for in the iron specification. All track timbers to receive three coats of red oxide of iron paint in linseed oil. Appropriate hand-railings to be provided at all bridges and on iron construction for the protection of trainmen using the sidewalks.

All materials used throughout must be the best of their respective kinds, all of the work to be done in the best, most substantial and workmanlike manner, according to the plans, drawings and these specifications, to the full intent and meaning of the same, and everything necessary for the proper and complete execution of the said plans and drawings, whether the same may have been particularly specified or not, or indicated on the plans referred to, to be done and furnished in a manner corresponding with the rest of the work, as well, as truly and as faithfully as though the same were herein particularly described and specially provided for.

Every part of the work to be executed under the direction and to the entire satisfaction, approval and acceptance of the engineer, and under the supervision of such agent as he may appoint.

Risks, Blame, Etc.—The contractor is to assume all risks and bear any loss occasioned by neglect or accident during the progress of the work, until the same shall have been completed and accepted by the Engineer. He is also to assume all blame or loss by reason of neglect or violation of state, city or district ordinances, laws or regulations, encroachments upon neighbors, or from any other cause.

Especial care must be taken, and every precaution used during the progress of the work, to protect the streets and to avoid injury or inconvenience to persons passing. The contractor must assume all responsibility for injury to persons or property.

Alterations.—It is understood that the company shall have the right to make any alterations, additions or omissions of work or materials herein specified or shown on the drawings, during the progress of the work, that they may find to be necessary, and the same shall be acceded to by the contractor or contractors, and carried into effect without in any way violating or vitiating the contracts.

The engineer shall have full power, at any time during the progress of the work, to reject any materials he may deem unsuitable for the purposes for which they are intended, or which are not in strict conformity with the spirit of these specifications. He shall also have the power to cause any inferior or unsafe work to be taken down and altered, at the cost of the contractor.

No charge shall be made by the contractor for hindrances or delay, from any cause, in the progress of any portion of the work in this contract, but it may entitle him to an extension of time allowed for completing the work sufficient to compensate for the detention, to be determined by the chief engineer, provided he shall give the engineer in charge immediate notice in writing of the cause of the detention.

Nor shall any claim be allowed for extra work unless the same shall be done in pursuance of a written order from the engineer in charge, and the claim made at the first settlement after the work was executed, unless the chief engineer, at his discretion, should direct the claim, or such part as he may deem just and equitable, to be allowed.

Any disagreement or difference between the company and contractor upon any matter or thing arising from these specifications or the drawings to which they refer, and all questions relating to the intent and meaning of the plans, drawings, specifications, or of the kind or

quality of the materials and work required thereby, shall be referred to the engineer, and his decision shall be final, conclusive and without appeal.

Parties making proposals for the iron work must furnish sample drawings, showing the general arrangement and design which they propose to use, and all designs and drawings must be subject to the approval of the chief engineer.

Proposals for the work will be made for the several structures erected, in place, complete, ready for use, and prices are to be named per foot lineal of the several structures, the length to count from center to center of end pins for open truss bridges, and from out to out for plate-girders. The masonry of abutments, piers and column foundations will be provided by the railroad company.

Bids must specify the earliest time of completion if the work is commenced forthwith.

The company does not bind itself to accept the lowest or any bid.

May 1st, 1891.

APPENDIX III.

SPECIFICATION FOR THE FURNISHING, MANUFACTURE AND ERECTION OF STRUCTURAL IRON WORK FOR THE TRAIN-SHEDS, ETC., OF THE PASSENGER STATION OF THE PHILADELPHIA AND READING TERMINAL RAILROAD COMPANY AT TWELFTH AND MARKET STREETS, PHILADELPHIA.

Several contracts may be made under these specifications for different parts of the work, in which case special instructions to bidders will be issued, with a list of drawings pertaining to the particular letting and explanatory thereof, which special instructions and drawings will be considered part of this specification.

Figured dimensions shall in all cases be taken in preference to scale measurements.

Additional Drawings.—Additional detail drawings will be furnished in exemplification of the work from time to time as they may be required, and it is to be distinctly understood that all such additional drawings shall be of equal force with those which are herein specifically cited, and the said additional drawings are to be considered as virtually embraced within and forming part of this specification.

General Conditions.—This specification is intended to embrace all the work as specifically stated in the several instructions to bidders, complete and ready for occupancy, the whole of said work to be comprised within this specification for any contract or partial contracts that may be made for the construction of the same.

All materials used throughout must be the best of their respective kinds, subject to the inspection and approval of the engineers and architects. The entire work provided for in this specification is to be constructed and finished in every part in a good, substantial and workmanlike manner, according to the accompanying drawings and this specification, to the full intent and meaning of the same, and everything necessary for the proper and complete execution of the plans and drawings, whether the same may have been herein particularly specified or not, or indicated in the plans referred to, to be done and furnished in a manner corresponding with the rest of the work, as well, as truly and as faithfully as though the same were herein particularly described and specifically provided for.

The general detail drawings of the structural iron work will be furnished to the contractor, who is to take the same and from them to make his own special shop drawings. These shop drawings must conform to the general detail drawings in every particular, and are to be submitted to the engineers and architects, and to be approved by them before the work is proceeded with; and said approval shall in no way authorize any change from the said general drawings, unless such change has been specially agreed to and made in the general drawings.

Four copies of each shop drawing are to be furnished to the engineers and architects for use in their office.

The contractor is also to verify all drawings furnished him, so far as relates to the dimensions and the interworking of the various parts, and he is to be responsible for the accuracy of his own shop drawings in all their details.

Every part of the work to be executed under the direction and to the entire satisfaction, approval and acceptance of the engineers and architects, and under the supervision and direction of such agents as they may appoint.

All questions relating to the intent and meaning of the plans, drawings, specification, or of the kind or quality of the materials and work required thereby, shall be referred to the engineers and architects, and their decision shall be final, conclusive and without appeal.

The contractor must be responsible for all patents, and must protect the railroad company against any and all claims on account of the use of the same.

Access must be allowed to the engineer and his assistants to the contractor's working drawings and shops for examination of details.

No charge shall be made by the contractor for hindrances or delay, from any cause, in the progress of any portion of the work in this contract, but it may entitle him to an extension of time allowed for completing the work, sufficient to compensate for the detention, to be determined by the engineers and architects, provided he shall give the engineer in charge immediate notice, in writing, of the cause of the detention.

In the case of subcontractors, the specifications are fully binding on them in every respect, and free access and information is to be given by them for thorough inspection of material and workmanship, and all required test pieces, etc., properly shaped, are to be provided as may be requested, without charge.

Any exceptional increase of cost of inspection of material due to location of works, must be borne by the contractor.

All shipments of material not properly inspected and passed are at the risk of the contractor.

The contractor must test the accuracy of the measurements as given by drawings with the actual foundations as provided on the ground for the structural iron work, so as to satisfy himself of their correctness before proceeding with the work.

The engineers and architects shall have full power at any time during the progress of the work to reject any materials they may deem unsuitable for the purposes for which they are intended, or which are not in strict conformity with the spirit of this specification. They shall also have the power to cause any inferior or unsafe work to be taken down and altered at the cost of the contractor.

Alterations.—It is understood that the company shall have the right to make any alterations, additions or omissions of work or materials herein specified or shown on the drawings, during the progress of the construction, that they may desire, and the same shall be acceded to by the contractor or contractors and carried into effect without in any way violating or vitiating the contracts. If any such changes are made, the value of the same shall be decided by the engineers and architects, who shall make an equitable allowance therefor, and shall add the amount of said allowance to the contract price for the work if the cost of the work has been increased, or shall deduct the amount from the contract price if the cost of the work has been lessened, as

they, the said engineers and architects, may deem just and equitable.

Also, that the railroad company will pay for no extra work or materials, unless ordered by them or their engineers and architects, with their approval.

Also, that any disagreement or difference between the company and contractor, upon any matter or thing arising from this specification, or the drawings to which it refers, or the kind or quality of the work required thereby, shall be decided by the engineers and architects, whose decision and interpretation of the same shall be considered final, conclusive and binding upon both parties.

Care of Finished Work.—Particular care must be taken by the contractor of all the finished work as the building progresses, which work must be covered up and thoroughly protected from injury or defacement during the erection and completion of the building.

Removal of Rubbish, Etc.—All refuse material and rubbish that may accumulate during the progress of the work must be removed from time to time, as may be directed by the agent in charge, and on the completion of the work, the building, streets and grounds must be thoroughly cleaned up, and the surplus material and rubbish removed.

Risks, Blame, Etc.—The contractor is to assume all risks and bear any loss occasioned by neglect or accident during the progress of the work, until the same shall have been completed, and accepted by the engineers and architects. He must properly protect any pavements of this and adjoining properties during the progress of the work, and make good any injuries that may have occurred to any adjoining building in consequence of or during the erection of this work. He is also to assume all blame or loss by reason of neglect or violation of city, district or state ordinances, laws or regulations, encroachments upon neighbors, or from any other cause.

Permits, Surveys, Etc.—The contractor must pay for all permits, surveys, inspectors' fees, or any other charges from borough, city, county or state officers.

Cast-Iron Work.

The cast-iron work to be true and sound, free from flaws and defects of any kind, the material good and tough and the lines corresponding exactly with the drawings. No rough or crooked castings will be accepted. All bolt and pin holes must be accurately drilled; all abutting surfaces must be planed, if necessary to insure even bearings and neat, close fittings, and the work must be properly fitted and erected.

Wrought Iron.

All wrought iron shall be of American manufacture.

All iron to be used in the tensile members of open trusses, laterals, pins, bolts, etc., must be double-rolled after and directly from the muck bar (no scrap will be allowed), and must be capable of sustaining an ultimate stress of 52 000 lbs. per square inch on a full section of test piece with an elastic limit of 26 000 lbs. per square inch and a minimum stretch of 20% measured after breaking in length of 8 ins.

All wrought iron not included under above head must be tough, fibrous, uniform in quality throughout, free from flaws, blisters and injurious cracks, and must have a workmanlike finish. It must be capable of sustaining an ultimate stress of 48 000 lbs. (excepting for plates over 24 ins. in width, which may be 46 000 lbs.) per square inch

on a full section of test piece, with an elastic limit of 26 000 lbs. per square inch.

All iron to be used in tension or subjected to transverse stress (except web-plates), must have a minimum stretch, on a length of 8 ins., of 15% measured after breaking.

All iron to be used in compression and for web-plates, of width not exceeding 24 ins., must have a minimum stretch of 10% on a length of 8 ins. measured after breaking.

All iron for web-plates exceeding 24 ins. in width, must have a minimum stretch of 5% measured in length of 8 ins.

When tested to the breaking, if so required by the engineer, the links and rods must part through the body and not through the head or pin hole. Such tests must be at the expense of the contractor when the requirements of these specifications are not complied with.

All tension wrought iron, if cut into testing strips 1½ ins. in width, must be capable of resisting without signs of fracture bending cold by blows of a hammer, until the ends of the strips form a right angle with each other, the inner diameter of the curve of bending being not more than twice the thickness of the piece tested. The hammering must be only on the extremities of the specimens, and never where the flexion is taking place. The bending must stop when the first crack appears.

All the tension tests are to be made on a standard test piece, of 1½ ins. in width, and from ½ to ¾ in. in thickness, planed down on both edges equally, so as to reduce the width to 1 in. for length of 8 ins. Whenever practicable, the two flat sides of the piece to be left as they come from the rolls. In all other cases both sides of the test pieces are to be planed off.

All plates, angles, etc., which are to be bent in the manufacture, must, in addition to the above requirements, be capable of bending sharply to a right angle, at a working heat, without showing any signs of fracture.

All rivet iron must be tough and soft, and pieces of the full diameter of the rivet must be capable of bending until the sides are in close contract, without showing fracture on the convex side of the curve.

Pins of 4½ ins. diameter or less may be rolled iron, but those of greater diameter must be forged.

All workmanship must be first class; all abutting surfaces, except flanges of plate girders, must be planed or turned, so as to insure even bearings, taking light cuts so as not to injure the end fibers of the piece, and must be protected by white lead and tallow. Abutting members must be brought into close and forcible contact when fitted with the splice-plates, and the rivet holes punched ¼ in. less in diameter and reamed in position before leaving the works; the plates being marked so as to go in the same position in erecting. Web-stiffeners shall be milled at the ends to make exact and tight fit to the flange angles.

The ends of all track stringers and other framed members are to be milled off to an exact length and in the direction required by the angle of the framing.

Generally the use of bolts instead of rivets will not be permitted, unless they are turned conical and the holes reamed to fit them.

Rollers must be turned and roller-beds planed.

Rivet holes must be carefully spaced and punched and must in all cases be reamed to fit where they do not come truly and accurately opposite, without the aid of drift pins. Rivets must be machine-driven wherever possible, and the machines must be such as will

retain the pressure on the rivet after the upsetting is completed. Rivets must completely fill the holes and have full heads, and be countersunk when so required.

Compression members must be straight and free from kinks or buckles in the finished piece.

Eye-bars and rods shall have no welds except in the case of forming loops for counters, laterals or sway rods. The heads of eye-bars and thickening for screw ends to rods must be made by upsetting, forging or rolling. Eye-bars must be perfectly straight before boring, the hole must be in the center of the head, and on the center line of the bar. Whenever links are to be packed more than $\frac{1}{8}$ in. to the foot of their length out of parallel with the axis of the structure, they must be bent with a gentle curve until the head stands at right angles to the pin in their intended position, before being bored; suitable blocking pieces being used to keep them in proper position during the operation of boring.

All pin holes in pieces which are not adjustable for length must be accurately bored at right angles to the axis unless otherwise shown in the drawings, and no variation of more than $\frac{1}{64}$ in. will be allowed in the length between centers of pin holes. All pieces must be at equal temperatures when bored, and those belonging to the same panel, when placed in a pile, must allow the pin at each end to pass through at the same time without forcing.

Pins must be carefully turned, perfectly finished and straight, and when driven in must have pilot nut to preserve threads. No variation of more than $\frac{1}{32}$ in. will be allowed between diameter of pin and pin-hole.

In the case of bolts a variation of $\frac{1}{16}$ in. will be allowed between diameter of bolt and hole.

Thickening washers must be used whenever required to make the joints snug and tight.

The contractor must punch all holes that may be required in the structural iron work for attachment of other portions of the structure.

PAINTING.

Material standing out before manufacture must be protected from rusting by pure raw linseed oil, applied when temperature of iron is 60° (Fahr.) or over. All inaccessible surfaces must be painted, before riveting up, with one heavy coat of pure red lead in pure raw linseed oil. All work after manufacture to be given one coat of pure raw linseed oil applied to iron of temperature of 60° or over, and thoroughly worked into all joints and openings. All parts that are inaccessible after erection must have two coats of pure red lead in pure raw linseed oil, at site before erection.

All iron after erection must have one good coat of pure red lead in pure raw linseed oil.

All iron to be scraped clean from scale before oiling or painting.

DISCUSSION.

Mr. Darrach. C. G. Darrach, M. Am. Soc. C. E.—The speaker said in amplification of the author's statements concerning the electrical installation, that the summer load on the incandescent dynamos is equivalent to 55 kilowatts, about 74% of the total load of one of the dynamos in the head-house, so that the plant is running quite economically.

The cold storage rooms are kept very free from moisture and vapor by reason of the numerous air locks. Eggs have been kept perfect, and sold and eaten as fresh after remaining 14 months in storage, and the speaker recently ate an apple which had been in cold storage for eight months and was perfect, without a sign of any defect whatever. On June 17th, 1895, the following temperatures were taken: Outside air, 76°; in the cold storage cellar surrounding the rooms, 68°; in the first air lock, 56°; in the second air lock, 50°; in the third air lock, 44°; in the fourth air lock or lobby to a freezing room, 34°; in the freezing room 3° Fahr. The temperature of the rooms is regulated to suit the materials stored; butter is kept at 20°, eggs and apples at a little above the freezing point, and meat at 33° to 34°.

During the summer, with brine at an initial temperature of 3° Fahr. the following ratio of brine cooling surface to net cubic contents maintains the temperatures mentioned in the rooms. The freezing rooms have side pipes in addition to the overhead pipes.

Cooling surface per cubic foot. Square feet.	Temperature, Fahr.	Kind of storage.	Remarks.
0.082	32°	Meats	Compartment room accessible to tenants.
0.065 to 0.04	29° to 33°	Meats	Transient, controlled by superintendent.
0.033	33°	Eggs	Long storage, controlled by superintendent.
0.033	33°	General	Long storage, controlled by superintendent.
0.057	20°	Butter	Long storage, controlled by superintendent.
0.27	3° to 4°	Freezing	Long storage, controlled by superintendent.

The 8-in. mains through which the brine circulates at a velocity of about 2 ft. per second lose only about 3° between the initial point and the return, although their length is about 1 200 ft., of which 600 ft. is in an atmosphere with a temperature of 95° to 100° Fahr.

CORRESPONDENCE.

Mr. Wagner. SAMUEL TOBIAS WAGNER, M. Am. Soc. C. E.—It must be remembered that at the time the train-shed described in the paper was constructed, it was the greatest single span train-shed in the world. As a roof, however, it had been eclipsed by the trusses of the Machinery Hall at the Paris Exposition of 1889, with their span of 364 ft. 2 ins., center to center of end pins, and by those of the Manufactures and Liberal Arts Building for the World's Columbian Exhibition at Chicago, with a span of 368 ft. center to center of end pins. Since its construction it has also been compelled to rank as second in the class of train-sheds by the roof of the Broad Street station of the Pennsylvania Railroad at Philadelphia.

The principal dimensions of the largest train-sheds in existence are as follows:

	Pennsylvania Railroad, Philadelphia.		Philadelphia and Reading Railroad, Philadelphia.		Pennsylvania Railroad, Jersey City.		Midland Railway, London, England.	
	Ft.	In.	Ft.	In.	Ft.	In.	Ft.	In.
Span of roof trusses, c. to c. end pins	300	8	259	8	252	8	240	0
Clear height at center	100	4	88	0	86	0	100	0
Width, out to out	304	0	266	6	256	0		
Length, out to out	598	0	506	8	652	6	600	0
Number pair of roof trusses	10		11		12			
Number of tracks	16		13		12		*10	

* Including 25 ft. roadway.

The practical construction of a roof of this magnitude was not undertaken without the most careful consideration and thought, especially as it was required that the chords of the trusses should be curved to the given radii instead of being straight between panel points, and it is believed that no other roof approaching it in magnitude has ever been constructed to the exact curves required by the drawings. Curves in shop construction are always extremely difficult to work with in order to insure accuracy in results, especially when in the same half truss there are five different radii to deal with, and compound curves in some of the individual sections into which the truss is divided for shipment. In order to insure the best workmanship and eliminate all chance of error, these conditions made it practically imperative

Mr. Wagner. that each half truss should be assembled by itself in the shop, all members being fitted to place and match marked. It will be remembered that in the case of the Forth Bridge in Scotland, practically the same method of procedure was adopted wherever the curved members of the truss occurred.

Curves in individual pieces offer no special difficulties, but whenever they form component parts of a whole, as in the case in discussion, the problem assumes a new and more serious aspect. Great care was observed in making the templets for the trusses and the best seasoned lumber was carefully selected and prepared. A complete duplicate set was made in case of accident and for the comparison of the working templets from time to time as the work progressed.

A space was prepared on the large floor of the bridge shop of the Phœnix Iron Company, covered with skids about 8 ft. apart, and generally radial to the chords of the truss. This space was sufficient to receive one-half of the truss. After the floor was prepared, the chord between the centers of pin-holes was calculated and laid off very carefully, the terminal points being established very accurately. From each of these points, by means of chords and deflection angles and using an engineer's transit, each panel point on the top chord of the truss was laid out, half of the truss being laid out from each end to insure accuracy. These lines checked within $\frac{1}{16}$ in. at the center panel point. The points were calculated to the back of the angles forming the top chord. After they were located the templets were placed on them and no discrepancy was found, thus forming a satisfactory check. Cast-iron chords with vertical lugs were then fastened to the skids, which were fastened solidly in place on the floor and the lugs set a given distance from the back of the chord angles. Hardwood blocks, bolted to these lugs, were used afterward to adjust for thickness of cover and splice plates. A series of points was thus established, which was used for reference in the future for each half truss. The method of procedure was follows:

Each half truss was divided into eight sections for shipment, marked from *A* at the heel of the truss to *H* at the peak. Sections *A* and *H* were riveted up and placed exactly in position on the skids. The chords of the remaining sections, which were of T shape, composed of a 12-in. vertical plate, two 6 x 6-in. angles and 14-in. covers, were riveted up, and all joints in the half truss faced but two; these joints were left unfaced for adjustment. The chords were then put in position on the skids, the top chords bearing against the hardwood blocks, and the diagonal and radial members fitted to place with the splice plates. All projecting cover plates over the splices on the tension chords were left blank and were drilled in place to insure good workmanship. No holes whatever were punched at the connections, where there was any likelihood of their not matching when assembled. All holes were then

reamed in place, and enough rivets driven by hand to insure against any motion when the truss was taken apart. Before taking it apart the pin holes were laid out and all loose pieces carefully match marked. Each section was then riveted by power, and finished in the usual way. Mr. Wagner.

On account of the curve required in the trusses, all web plates forming the chords had to be curved on edge. This was done on the straightening plate of the rolling mill, the straight edge on this plate being replaced by wrought-iron forms forged to the proper radii. This was done so successfully with the initial heat from the rolls, that no further work was necessary. The angles were all curved cold under a cam press. In some cases it became necessary to slightly recurve them after they were punched. No special work was required upon the purlins or the side trusses; the jack rafters, however, having the same curve as the top chord of the main trusses, were rather more difficult to handle, on account of the necessity of having the open holes for attachment to the purlins at the proper angle. These were assembled in the usual way and the holes for these connections set, in each case, by full-length templets. The fact that there was no trouble in the field in making any of these difficult connections showed the wisdom of this method of construction.

The facilities for erection were excellent. The plant consisted of a falsework upon which the arch was assembled, and a traveler framed together to make a single machine. The falsework extended the entire inside width of the train shed, and filled the space completely between the floor and the lower chord of the trusses. It was 17 ft. 9 ins. wide, center to center of legs at the base, and 5 ft. at the top of the cap, thus giving sufficient width to assemble the pair of trusses with their bracing upon the top. The legs consisted of 6 x 6-in. timbers, thoroughly braced in both planes by 2 x 8-in. and 2 x 6-in. pieces. Fourteen lines of rails ran longitudinally through the train-shed, and the falsework rested upon them by means of wheels 18 ins. in diameter, thus enabling the whole structure to be moved longitudinally. The traveler, about 126 ft. in total height and 40 x 28 ft. in plan on the level of the floor, was arranged as part of the falsework and rested upon the timbers forming its base. It was capable of motion in a transverse direction, and, as a part of the whole erection machine, could be moved longitudinally simultaneously with the falsework. It was provided with an overhanging top, which spanned two pairs of trusses, and was counterbalanced on the lower platform by means of two hoisting engines, their boilers, and 45 000 lbs. of ballast to prevent any overturning. The main posts were 8 x 8 ins., the back stay was 6 x 8 ins., with two rods 1¼ ins. square. The bracing was 2-in. plank, 6 to 8 ins. in width, thoroughly fastened.

The method of operating the falsework and traveler was as follows:

Mr. Wagner. The sections of the arch trusses were erected in pairs and braced together as fast as they were put in place, the connections being bolted. This operation was continued until the peak was reached, when the top pin was driven, completing the whole span. The purlins and jack-rafters, with the ventilator frame, were then placed in position, and riveting begun when all the joints were in place and had their full bearing. The time required to move the falsework from one pair of trusses to another, a distance of 50 ft. 2 ins., was about 40 minutes. The construction and erection of this shed was done by the Phœnix Bridge Company.

Mr. Wilson. JOSEPH M. WILSON, M. Am. Soc. C. E.—The author desires to state, in further explanation of the construction of the head-house, that the lower portion of the building, from the nature of things, almost necessarily became a fire-proof construction, the apartments being very large and the floors paved with marble or cement. The upper part of the structure is really something more than "of slow-burning construction" as that term is generally understood. The carrying framework is entirely of iron and steel, protected from the effects of fire by mackite, and the floors have no wooden girders, but consist of 3-in. planks, tongued and grooved, fastened to nailing strips on rolled steel beams and finished with 1-in. flooring boards on the top, the whole being thoroughly protected underneath by mackite. The partitions are of solid mackite. It will be seen that the only exposed wood is the upper surface of the floor which is required in the office rooms, the sash, and the doors and frames. The entire construction can therefore almost be classed as fireproof.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

758.

Vol. XXXIV.—September, 1895.

CONSUMPTION AND WASTE OF WATER.

By DEXTER BRACKETT, M. Am. Soc. C. E.

READ AT THE ANNUAL CONVENTION, JUNE, 1895.

WITH DISCUSSION.

An examination of the annual reports of the water departments of the cities of the United States shows that the quantity of water used has increased much more rapidly than the growth of the population, and much more rapidly than had been expected by the engineers who designed the works. Take the case of Boston, which may be considered typical of many other cities. The original works were constructed in 1848, and were based upon the assumption that 28 galls. per capita would be an adequate supply for all purposes, this conclusion being based on the experience of the city of Philadelphia. Before many years it was discovered that 28 galls. per capita were not sufficient, and in estimating for an additional supply in 1872, J. P. Davis, M. Am. Soc. C. E., considered that 60 galls. for each person would be a proper allowance for future needs. The present consumption in Boston is about 100 galls. per inhabitant; the capacity of the supply obtained

TABLE No. 1.

Daily Average Consumption of Water in Gallons per Capita per Day in Various Cities in the United States.

	Years.	Boston, Cochituate Works.	Boston, Mystic Works.	Chicago.	Philadelphia.	Brooklyn.	St. Louis.	Cincinnati.	Cleveland.	Detroit.
Population.	1850	139 800	.....	29 963	121 376	96 838	77 860	115 435	17 034	21 019
	1860	177 900	.....	112 172	565 529	266 661	160 773	161 044	43 417	45 619
	1870	225 100	87 071	298 997	674 022	396 099	310 864	216 039	92 829	79 577
	1880	306 000	107 700	503 185	847 170	566 663	350 518	255 139	160 146	116 340
	1890	410 130	117 500	1 099 850	1 046 964	806 343	451 770	296 908	261 353	205 876
Daily Average Consumption.	1850	42	.....	.....	.....	.....	.....	20	.....	.....
	1855	64	.....	.....	.....	.....	.....	25	.....	44
	1860	97	.....	43	36	.....	.....	30	14	52
	1865	66	27	42	50	29	.....	29	22	55
	1870	66	44	73	55	47	.....	48	31	64
	1871	60	56	72	55	47	.....	53	36	76
	1872	63	70	74	54	53	45	54	40	88
	1873	72	77	88	56	59	51	50	43	98
	1874	72	73	96	58	54	55	55	45	97
	1875	69	86	100	69	59	61	60	44	120
	1876	71	80	103	.....	57	62	68	49	111
	1877	72	75	.....	58	59	66	64	56	111
	1878	80	76	123	64	58	67	66	51	110
	1879	87	88	.....	66	60	72	68	63	125
	1880	87	87	112	68	54	72	76	65	130
	1881	94	80	.....	71	56	76	87	77	145
	1882	95	73	110	76	58	76	69	68	132
	1883	97	74	.....	76	58	75	66	76	146
	1884	73	65	114	74	61	63	74	83	159
	1885	73	68	116	72	64	67	64	93	176
	1886	74	72	118	80	65	73	74	91	176
	1887	80	72	120	89	65	73	88	96	197
	1888	87	75	119	100	67	74	99	95	204
	1889	81	69	123	110	67	73	99	99	172
	1890	83	71	127	132	68	78	115	106	155
	1891	90	75	135	140	70	83	138	111	144
	1892	96	79	134	143	79	89	123	118	140
	1893	107	86	147	150	86	95	124	130	148
	1894	101	89	.....	.....	.....	.....	.....	.....	144

TABLE No. 1—(*Continued*).

	Years.	Milwaukee.	Louisville.	Providence.	Lowell.	Fall River.	Cambridge.	Lynn.	New Bedford.	Salem.
Population.	1850	20 061	43 194	41 513	33 383	11 524	15 215	14 257	16 443	20 264
	1860	45 246	68 033	50 666	36 827	14 026	26 060	19 083	22 300	22 252
	1870	71 440	100 753	68 904	40 928	26 766	39 634	28 233	21 320	24 117
	1880	115 587	123 758	104 857	59 475	48 961	52 669	38 274	26 845	27 563
	1890	204 468	161 129	132 146	77 696	74 308	70 028	55 727	40 733	30 801
Daily Average Consumption.	1850	.....	.....	.....	.....	.....	.....	.....	.....	.....
	1855	.....	.....	.....	.....	.....	.....	.....	.....	.....
	1860	.....	.....	.....	.....	.....	.....	.....	.....	.....
	1865	.....	18	.....	.....	.....	.....	.....	.....	.....
	1870	.....	23	.....	.....	.....	44	.....	.....	31
	1871	.....	21	.....	.....	.....	43	.....	.....	38
	1872	.....	22	.....	.....	.....	38	.....	.....	45
	1873	.....	22	.....	.....	.....	48	41	.....	55
	1874	.....	24	.....	.....	12	50	42	.....	51
	1875	29	24	.....	24	18	57	40	.....	.....
	1876	45	27	.....	29	25	51	32	.....	59
	1877	69	29	24	30	26	53	32	.....	58
	1878	85	30	26	32	25	45	32	.....	53
	1879	96	33	30	35	27	47	34	.....	55
	1880	106	42	34	38	28	46	32	.....	55
	1881	109	56	35	40	30	46	32	86	59
	1882	114	47	33	43	36	45	37	82	63
	1883	108	52	37	46	31	47	36	78	61
	1884	101	56	34	48	26	46	39	72	60
	1885	105	62	37	56	26	51	42	85	62
	1886	110	65	37	59	27	56	44	86	64
	1887	117	64	37	62	25	60	48	85	77
	1888	105	62	40	69	28	63	45	88	71
	1889	116	67	41	62	27	62	43	89	68
	1890	114	71	46	69	28	65	45	98	69
	1891	112	78	48	74	30	67	49	92	87
	1892	101	81	57	73	27	71	53	88	81
	1893	108	75	63	79	27	80	54	91	66
	1894	.....	.....	.....	75	.....	70	.....	.....	.....

in 1872 will soon be reached, and the Massachusetts State Board of Health, in a report on the water supply for the Boston Metropolitan District, has come to the conclusion that 100 galls. per capita is a proper allowance for the future needs of the district which embraces all the cities and towns within 12 miles of the State House.

Although the present consumption in Boston is about four times as large as the estimate of the designers of the works, it is still less than that of a number of large American cities. The table on pages 186 and 187 gives the per capita consumption in some of the cities of the United States and shows the great increase which has taken place, especially during the past five years.

The question naturally arises, what are the causes of this great and constantly increasing use of water? Is the increase due to a legitimate use, or is it due to a wasteful and extravagant use of the water? The author is of the opinion that the increase can be attributed in part to both of these causes, but that the greatest cause is the unnecessary and wilful waste.

With a few exceptions the efforts of the authorities in charge of water supplies during the past 25 or 30 years have been directed toward the securing of new sources of supply and the construction of additional works, rather than toward the promotion of measures of economy in the use of the supplies already provided. Public opinion has always favored the free use of water, and in most cities there exists a well-established but mistaken idea that constant streams of water running through water-closets and house drains have a tendency to prevent or remove the formation of noxious gases and disease germs.

In the following paper an attempt has been made to study the causes of the great increase in the consumption of water, with the object of learning: *First*, what can be considered a proper allowance per capita in designing works for future supply? *Second*, what are the causes of waste and how can it be best prevented?

Consumption per Capita.—In considering the proper allowance to be made for the use of water, it will be convenient to consider the question under the following subdivisions:

Quantity used for domestic purposes.

Quantity used for trade, manufacturing and mechanical purposes.

Quantity used for public purposes.

Quantity wasted.

Quantity Used for Domestic Purposes.—Under the first head should be included not only the household uses of the inhabitants, but also the quantity required for provision stores, fish markets, laundries and stables, in fact, all the requirements of a residential community, with the exception of the public uses of fountains, street watering, fires, etc.

TABLE No. 2.

CONSUMPTION PER CAPITA IN BOSTON, BROOKLINE, NEWTON, FALL RIVER, WORCESTER, YONKERS AND LONDON, AS DETERMINED BY METER MEASUREMENT.

CITY OR TOWN.	Number of houses.	Number of families.	Number of persons.	CONSUMPTION, GALLONS PER—		REMARKS.
				Family.	Capita.	
Boston	31	402	1 461	221	59	Highest cost apartment houses in the city.
Boston	46	628	2 524	185	46	First-class apartment houses.
Boston	223	2 204	8 432	123	32	Moderate class apartment houses.
Boston	39	413	1 844	80	16.6	Poorest class apartment houses.
Boston	339	3 647	14 261	139	35.6	Average of all apartment houses supplied by meter.
Boston	40	....	1 699	....	46.1	Boarding houses.
Brookline	..	828	4 140	221.5	44.3	Average of all dwellings supplied by meter.
Newton	490	490	2 450	132.5	26.5	All houses supplied with modern plumbing.
Newton	....	619	3 005	....	6.6	These families have but one faucet each.
Newton	....	278	1 390	34.5	6.9	These families have but one faucet each.
Fall River	28	34	170	127.5	25.5	The most expensive houses in the city.
Fall River	64	148	740	42.0	8.4	Average class of houses, generally having bath and water-closet.
Fall River	....	....	70 000	....	12.3	Total domestic consumption.
Worcester	....	20 514	90 942	....	16.8	Whole domestic consumption.
Worcester	....	81	327	80.2	19.9	Woodland street, best class of houses.
Worcester	....	37	187	118.1	23.4	Cedar street, best class of houses.
Worcester	....	93	447	95.0	19.8	Elm street, houses of moderate cost.
Worcester	....	245	1 104	55.1	12.2	Southbridge street, cheaper houses.
Worcester	....	229	809	55.0	15.6	Austin street, cheaper houses.
Yonkers, N. Y.	....	....	31 000		21.4	
London, England.	1 169	....	8 183	.....	25.5	Houses renting from $250 to $600; each have bath and two water-closets.
London, England.	727	....	5 089	.....	18.6	Middle class, average rental, $200.

The records obtained in cities and towns where water meters are used on house services furnish the best information obtainable in regard to the actual quantity needed for domestic purposes. Table No. 2 embodies the results of data collected, showing the actual con-

sumption per capita for household or domestic use by different classes of people in a number of cities, as determined by meter records.

There appears to be a marked difference in the household consumption in different places. In the case of Brookline, the average per capita consumption of 44.3 galls. includes the amount used for private stables, conservatories, lawns, etc., and the use for these purposes is very much larger in that town than in the average suburban community.

In Newton the consumption of 26.5 galls. per capita represents the quantity used by people residing in modern houses valued at from $5 000 to $15 000 and containing all the modern plumbing conveniences, but the houses are on smaller estates than many of those in Brookline and the number of stables and conservatories is much smaller.

The houses in Worcester and Fall River are, as a whole, much less expensive and have fewer water fixtures than those in Brookline and Newton. The figures given for London are taken from the Report of the Royal Commission on the Water Supply of the Metropolis. The consumption per capita in English cities is generally much smaller than in the United States, but these figures indicate that the actual quantities used in buildings having water fixtures of similar character are the same there as in our cities.

The per capita consumption in Newton, Fall River, Worcester and London, in first-class houses supplied with modern plumbing, was in each case about 25 galls. In two instances, Worcester 16.8 galls., and Fall River 12.3 galls., the figures given embrace not only the household consumption, but all domestic uses, including small stores, groceries, laundries and stables, in fact, every use with the exception of manufacturing, mechanical and public uses.

In Yonkers, N. Y., all of the water is supplied through meters, except for builders' use, street sprinkling, public buildings and street and sewer use. The number of consumers in 1894 was estimated to be 31 000, and the records of the meters show that the domestic consumption was 21.4 galls. per consumer.

It is worthy of notice that the averages in the table are not based upon the records of a few families. Combining all of the records, there is a population of 232 000 using 18.2 galls. per capita. With an addition of 10% to cover loss in the registration of the meters, the consumption per person would be but 20 galls.

TABLE No. 3.

TABLE SHOWING CONSUMPTION IN GALLONS PER CAPITA FOR VARIOUS PURPOSES FROM THE COCHITUATE WORKS IN BOSTON IN 1880 AND 1892.

	POPULATION, 1880, 306,000.			POPULATION, 1892, 430,000.		
	Metered.	Unmetered.	Total.	Metered.	Unmetered.	Total.
MANUFACTURES AND TRADE.						
Office buildings and stores	0.844	10.200	11.044	5.63	5.54	11.17
Steam railroads	1.139	.....	1.139	2.26	.....	2.26
Sugar refineries	0.811	.....	0.811	1.70	.....	1.70
Factories, machine shops, mills and engines	0.966	2.120	3.086	2.15	.....	2.15
Iron works and foundries	0.573	.....	0.573	0.24	.....	0.24
Marble and stone works	0.143	.....	0.143	0.12	.....	0.12
Gas companies	0.324	.....	0.324	0.75	.....	0.75
Electric light companies	.....	.....	.....	0.69	.....	0.69
Breweries	0.556	.....	0.556	0.89	.....	0.89
Oil and chemical works	0.214	.....	0.214	0.19	.....	0.19
Laundries	.....	0.150	0.150	0.15	0.35	0.50
Restaurants	0.129	0.650	0.779	0.37	0.29	0.66
Stables	0.443	.....	0.443	0.60	.....	0.60
Steamers and shipping	0.325	1.000	1.325	0.82	0.08	0.90
Elevators and motors	1.033	.....	1.033	2.95	.....	2.95
Street railways	.....	.....	.....	0.90	.....	0.90
Saloons	.....	1.500	1.500	0.27	0.89	1.16
Hotels	1.454	0.150	1.604	1.55	0.07	1.62
Theatres and halls	.....	.....	.....	0.10	0.09	0.19
Markets and cellars	.....	0.150	0.150	.....	.....	.....
Greenhouses	.....	0.160	0.160	.....	0.08	0.08
Miscellaneous	0.314	.....	0.314	0.27	0.28	0.55
Totals	9.268	16.080	25.348	22.60	7.67	30.27
DOMESTIC USES.						
Apartment hotels	0.047	5.850	5.897	1.72	12.34	14.06
Dwelling houses	.....	50.000	50.000	.....	43.90	43.90
Stables	.....	1.500	1.500	.....	1.38	1.38
Hand hose	.....	1.250	1.250	.....	2.25	2.25
Club houses	.....	0.040	0.040	0.18	0.07	0.25
Churches	.....	0.250	0.250	.....	0.18	0.18
Miscellaneous	.....	.....	.....	.....	0.22	0.22
Totals	0.047	* 58.890	58.937	1.90	* 60.34	62.24
PUBLIC USES.						
Hospitals	.....	0.200	0.200	0.30	0.21	0.51
Schools	0.505	0.400	0.905	0.30	0.12	0.42
City, State and Government buildings	.....	0.400	0.400	0.83	0.52	1.35
Urinals, fountains, etc	.....	.....	.....	.....	0.14	0.14
Miscellaneous	.....	0.500	0.500	.....	.....	.....
Totals	0.505	1.500	2.005	1.43	0.99	2.42

* The quantity wasted is included in the unmetered consumption.

Quantity Used for Mechanical, Trade and Manufacturing Purposes.—The quantity of water required for mechanical, trade and manufacturing purposes varies very largely in different communities. In some cities and towns there is very little manufacturing and almost the en-

tire consumption is for domestic use; in other places the domestic consumption is less than the quantity used for manufacturing purposes. Even in manufacturing cities conditions often exist which affect the demand for manufacturing purposes upon the public water supply Fall River is a manufacturing city, but most of the mills obtain a supply of water from the river, and the total amount used in the city for manufacturing purposes is but 2 galls. per inhabitant. On the other hand, where the manufacturers are obliged to take their supply from the public works, the use for manufacturing and trade purposes is from 20 to 30 galls. per inhabitant.

The present consumption in Boston for these purposes is about 30 galls. per inhabitant, and as most of this water is supplied through meters, the quantity used for different purposes can be determined. Table No. 3 shows the consumption in gallons per capita for various purposes from the Cochituate works in Boston in 1880 and also in 1892. The quantity used and paid for by schedule rates has been estimated by dividing the total amount received in proportion to the amount received for different purposes. This method gives results which are fairly accurate, and as the proportion of unmetered water used for manufacturing and trade purposes is small, the quantities used for the purposes enumerated in the table are thought to be nearly correct.

The use of water for mechanical and trade purposes in Boston is increasing at a much more rapid rate than the population, and the author sees no reason why the conditions existing in Boston should not exist in other cities. A very forcible illustration of this fact is given by the following statement showing the consumption per inhabitant for six of the more important mechanical and manufacturing uses in 1880 and 1892. All of the water used by these classes of takers is metered.

Gallons per Capita.	1880.	1892.
Steam railroads	1.14	2.26
Sugar refineries	0.81	1.70
Gas works	0.32	0.75
Electric light and power	.....	0.69
Breweries	0.56	0.89
Elevators	1.03	2.95
Totals	3.86	9.24

The daily quantity used in 1880 for the above purposes was 1 242 900 galls., and in 1892 it was 3 973 200 galls., a difference of

2 731 000 galls., or more than 20% of the total increase in the consumption of the works.

Quantity Used for Public Purposes.—Public purposes include those for public buildings, hospitals, institutions, schools, street sprinkling, flushing sewers, ornamental and drinking fountains, fires, etc. The quantity of water used for these purposes varies to a great extent in different cities, on account of the difference in the methods of supply. In some cases no charge is made for water used for public purposes, while in others the water is supplied through meters and the water department is paid for all water used. When no charge is made the consumption is apt to be excessive. The following examples illustrate this point: In Detroit one hospital containing 150 persons used 625 galls. per capita; another, containing 135 persons, used 548 galls. per capita. A school consumed 2 112 000 galls. in one month, but on charging by meter measurement the consumption was reduced to 435 000 galls.

In Boston many of the public buildings are metered, and the quantity used for public purposes is more nearly the quantity actually needed. The metered consumption for public buildings, schools and hospitals in 1892 was 1.5 galls., and the unmetered quantity is estimated at 0.8 gall., making 2.3 galls. per capita used for these purposes. A careful study of the quantity required for different public uses, based upon such information as the author has been able to obtain, gives the following requirements for the different purposes:

	Gallons.
Public buildings, schools and hospitals	2.30
Street sprinkling	1.00
Flushing sewers, public urinals	.10
Ornamental and drinking fountains	.25
Fires	.10
Total for public purposes	3.75

Probably 4 or 5 galls. per capita should cover all the requirements for public purposes.

From a study of the preceding facts the author draws the following conclusions in regard to the actual requirements of different communities: The quantity needed for domestic use is not more than 30

galls. per inhabitant, and in communities where the number of water fixtures is small in proportion to the population supplied, a smaller quantity will answer all requirements. For business, mechanical and manufacturing use, the amount per capita will differ very largely in different cities, and for various reasons. It is not probable, however, that the actual requirement at present exceeds 40 galls. per capita in any of our large cities.

The quantity needed for public use is not more than 5 galls, making a total of 75 galls. as the maximum quantity needed for actual use, without any allowance for waste.

Quantity Wasted.—If the statements which have been made as to the actual amounts required for domestic use, for manufacturing, mechanical and trade purposes, and for public uses are correct, it is evident that a very large proportion of the total consumption of most of the large cities and towns must be wasted, and in the quantity wasted the author does not include water which is lavishly used, but only that which is either negligently or wilfully permitted to escape without doing any useful service.

Without doubt, the local conditions affect to some considerable degree the per capita consumption for trade, mechanical and public purposes, but making liberal allowance for these conditions, the difference between 75 galls., which should amply supply the legitimate demands of a large city like Boston, and the quantity now used, as given in Table No. 1, shows that the amount wasted in many cities must be more than that used.

There are a number of ways by which the existence and amount of the waste can be quite accurately determined, one of which is by the use of the Deacon waste water meter. These meters have been in use in Boston since 1880, where they were introduced by J. P. Davis, M. Am. Soc. C. E., then city engineer. This meter is placed upon the main supply of a district of from 2 000 to 3 000 persons, and furnishes a continuous record of the rate of consumption in that district. The records of these meters show conclusively that there is an enormous consumption of water during the night, when the legitimate use should be small. In 1883, when these meters were first placed in general use, the average daily consumption in a residential district of 34 800 persons was 69.8 galls. per capita, and between the hours of 1 and 4 A. M., when the legitimate use should be the least, the consump-

tion in the same district was at the rate of 53.2 galls. per head per day. The minimum rate was nearly 80% of the average for the entire day.

During the week ending August 20th, 1893, observations were made to determine the rates of consumption for different portions of the 24 hours from the Mystic works, which supply a population of about 117 000. The results were as follows:

RATE PER HEAD PER DAY.

Time.	Gallons.
1 A.M. to 4 A.M.	40.8
4 " " 7 "	58.6
7 " " 10 "	103.8
10 " " 1 P.M.	93.0
1 P.M. " 4 "	93.2
4 " " 7 "	79.5
7 " " 10 "	61.9
10 " " 1 A.M.	52.9
Average	73.6

In this case the minimum rate was about 60% of the average and 40% of the maximum, and it is probable that at least 30 galls. per day were wasted.

If there is shown to be a waste of 30 galls. per capita where the total daily consumption is but 74 galls., what must be the condition of affairs in some of the large cities where the consumption is from 125 to 250 galls. per inhabitant? In Buffalo the daily average consumption for the year 1894 was 72 270 000 galls. or 252 galls. per inhabitant. L. H. Knapp, M. Am. Soc. C. E., the engineer of the Buffalo works, states that 70% of the water which they pump is estimated to run to waste, and that it is hard to tell where they will land or how this enormous use of water is to be prevented. The people are not in favor of meters. They have reached the capacity of the works, both as to tunnels, engines and pipes. The additional cost of pumping in Buffalo due to this waste, to say nothing of the increased interest and maintenance account, is from $75 000 to $100 000 per annum. In Chicago, Cleveland, Philadelphia and Detroit, the consumption is about 150 galls., of which probably one-half is wasted.

This waste can be attributed to the following causes: 1st, defective plumbing; 2d, defective street mains and services; 3d, waste to prevent freezing of services; 4th, wilful waste.

First. Defective Plumbing.—In cities where meters are not generally used, defective house plumbing is the cause of the greater portion of the waste of water. The general use of all the modern plumbing conveniences, even in the cheaper class of houses, has become the custom, and has resulted in the manufacture and sale of a cheap class of water fixtures which are the cause of a very large proportion of the waste. From 10 to 15% of the premises examined in Boston are found to have defective fixtures, and of 12 609 defective fixtures reported in 1893, 7 314 were ball cocks. The tank ball cock is without doubt the source of the greater part of the waste from defective plumbing. Unseen and unnoticed a constant and oftentimes large stream runs silently down the overflow pipe to the sewer, and as the water bill is not increased, the property-owner finds it cheaper to let it run than to call in the plumber, the sight of whose bills he fears more than the sound of running water or the complaints of his tenants.

Second. Defective Mains and Services.—From a careful study of the question the author is of the opinion that the waste from the street mains and services is much larger than is generally supposed. It is often said that no leak of any magnitude is likely to occur without showing on the surface of the ground, but broken 4-in. and 6-in. pipes and service pipes running 1 000 galls. per hour have been discovered in Boston which gave no indication at the surface, and which had evidently been wasting water for years. Only a short time ago one of the waste water meters showed an excessive consumption in a short street supplied by a 6-in. main, and a careful investigation disclosed the fact that the main was broken entirely off and the water was escaping unseen into the sewer at the rate of 100 000 galls. in 24 hours.

Third. Waste to Prevent Freezing of Services.—This cause of waste, while not continuous throughout the year, is very large during seasons of extreme cold. The average daily consumption of the Cochituate and Mystic works for the months of November, 1892, and March, 1893, was 51 924 600 galls., while for the intervening winter months it was 60 149 600 galls. The difference of 8 225 000 galls. per day, equivalent to 3 380 000 galls. per day or 6 galls. per capita for the entire year, may be properly charged to cold weather waste.

Fourth. Wilful Waste.—The great cause of wilful waste is the deliberate fastening open of water-closets, thus permitting a constant flow of water for the purpose of cleansing the drains. The desired

result is seldom if ever accomplished, but the ignorant water-taker cannot be convinced of the futility of the plan, and the department suffers in consequence. An example of the wilful use or misuse of water has been brought to the author's notice where a constant stream of water flowing through the kitchen sink was made to answer the purpose of a refrigerator.

Prevention of Waste.—There is no doubt that the most efficient means of preventing waste of water on the premises of the water-takers is the universal use of water meters, for if every water-taker is obliged to pay for all the water entering his premises whether used or wasted, the quantity wasted is very sure to be reduced to a minimum.

The argument has been advanced that the use of meters tends to reduce the consumption of water for sanitary purposes below a point conducive to public health, but this argument has little foundation in fact. The great cause of waste of water is the continual small trickling streams which are of no practical benefit in cleansing drains. The introduction of meters has always been followed by a reduction in consumption, and in the cities and towns where their use has been adopted with the construction of the works, the quantity required per capita is universally very low.

The consumption in the following cities shows the effect of this method of waste prevention:

	Consumers.	Gallons per consumer.	Percentage of services metered.	Metered consumption. Gallons per consumer.
Fall River	85 576	28.5	80.	....
Woonsocket	20 000	26.1	85.2	....
Yonkers—1894	31 000	85.	99.4	52.2
Wellesley—1894	2 933	63.	91.	27.4
Newton—1894	28 500	57.	76.	29.2

By the adoption of a minimum rate which provides an ample allowance for all domestic purposes, the objection to the use of meters on sanitary grounds is obviated. In some cities where the citizens have become accustomed to the lavish and wasteful use of water, and where the mistaken idea is prevalent that water should be as free as the air, the opposition of public opinion will not sanction the general introduction of meters. The people will authorize the expenditure of millions for new supplies of water, larger mains and additional pumping ma-

chinery, but they will strenuously oppose the introduction of meters. In these places much can be done by a thorough and constant house-to-house inspection.

In the author's opinion, next to the general use of meters, the most efficient method of preventing waste is by house-to-house inspection aided by the Deacon waste-water meter. This system is largely used in England, but Boston is the only city of the United States where it has been adopted.* The Deacon waste water meter is the invention of Mr. G. F. Deacon, and was first used in Liverpool about 20 years ago. A trial of the meter in Boston in 1880 gave such satisfactory results that the system was adopted and has been in constant use since 1883. As the construction and operation of this meter is entirely different from the ordinary recording meter, the following description is given:

The cast-iron body of the meter contains a hollow, cone-shaped chamber of brass (see Fig. 1). Suspended in this chamber C is a composition disc D of the same diameter as the upper end of the cone. The fine wire by which the disc is suspended passes through a small hole in the top of the chamber over the pulley R, and is attached to a counter-weight W. When the water in the pipe is at rest the disc is held at the top of the cone by the counter-weight, but when water is drawn through the meter the disc is pressed downward, its position depending upon the quantity of water passing through the meter.

By means of a pencil attached to the wire, the motion of the disc is recorded upon the drum E, revolved by clockwork once in 24 hours. The meter does not, like the ordinary house meter, record the total quantity which has passed between any two observations, but it records upon a diagram the rate of flow at each and every instant. It is thus possible to ascertain the rate of consumption during the night when the legitimate use of water is very small, and determine the amount of water wasted. The meters are placed under the sidewalk upon a by-pass leading from the main, so arranged that by closing the valves on the street mains all of the water used in the section to be tested is drawn through the meter.

The meters in use in Boston are 6 ins. in size, and have a registering capacity of 25 000 galls. per hour. A population of from 3 000 to 6 000 can be supplied at one time through the meter, and the arrange-

* A few meters have been used in Philadelphia for experimental purposes, but the system has not been adopted for general use.

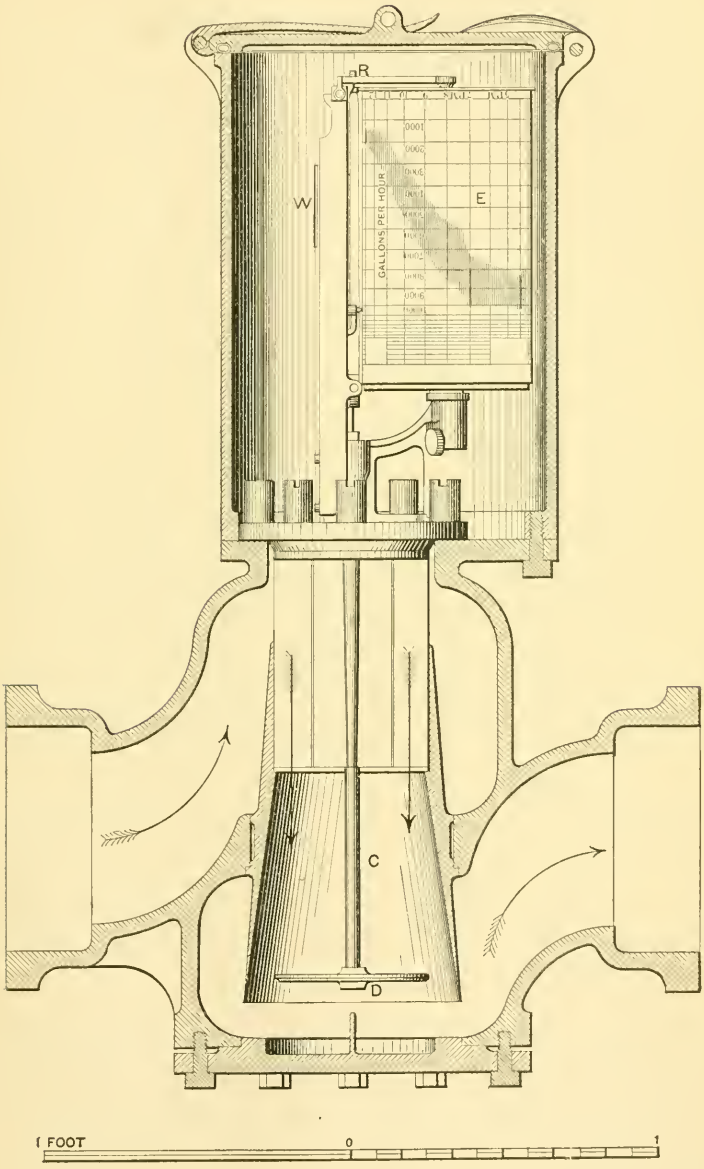


FIG. 1.

ment of the meters and pipe system is such that each meter is used to supply two or more sections. The number of meters now in use is 83, supplying a population of about 420 000.

The method of detecting and preventing waste by this system is as follows: The valves on the street mains are closed, so that all the water in the section to be tested is drawn through the meter. One or two diagrams are obtained showing the rate of consumption, after which the night inspection is begun. The ordinary iron wrench used to let on or shut off the house water supply is applied by the inspector to the sidewalk stop-cock, and by placing his ear to the wrench he can determine whether any water is passing through the service. Note is made of all premises where there are indications of waste, and a complete examination is made by a day inspector to locate the cause of the waste. Notices are then issued to the owners or occupants of the premises, and all defective fixtures are repaired.

One important advantage derived from the use of these meters is that the work of inspection is done where most needed. The meter records determine the streets where the waste is taking place, and the work of the inspector can be concentrated where the greatest saving can be made. Another benefit is the detection of waste from the street mains and services. Neither the use of meters nor the ordinary system of house-to-house inspection will prevent the waste from this cause, and that it is much larger than is generally supposed, the following facts will serve to illustrate.

In the city of Yonkers in 1894, 32 000 consumers were supplied with water through 3 504 services, on which there were 3 482 meters. The daily average quantity pumped was 2 623 760 galls., or 85 galls. per consumer, while the total quantity registered by the meters was 52.3 galls. per consumer; of the remainder, 32.7 galls. per consumer or 1 010 000 galls. per day, a small portion was used for public purposes, but the great proportion is unaccounted for.

In Milton, Mass., all water used in the town is measured by a meter on the supply main, and every service pipe is provided with a meter. The works are but four years old, have 18 miles of cast-iron mains, 376 services supplying about 2 300 persons, and with the exception of water used for flushing mains, street sprinkling, street construction and for fires, all of the water used is measured by meters on the service pipes. The daily average amount registered by the meter on

the supply main in 1893 was 128 529 galls., while the total recorded by the service meters was 65 842 galls. Allowing 2 000 galls. per day for blowing off pipes and fires, there remains 62 687 galls., or nearly 50% of the total consumption which is unaccounted for. In Newton and Fall River., Mass., and Woonsocket, R. I., similar results are shown.

In the city of Newton all services, with the exception of houses having a single faucet, are metered ; and yet, making a liberal allowance for the amount used by the unmetered takers, the quantity of water delivered into the premises of the water-takers during the year 1892 was but little more than one-half of the total quantity pumped. Of a daily average consumption of 1 288 000 galls., 595 600 galls. were used for public purposes or wasted from the street mains or services, or passed through the meters without being recorded.

In the city of Fall River during 1892, 2 217 370 galls. per day, equivalent to 27.24 galls. per capita, were pumped into the mains ; and 1 274 500 galls., or 15.66 galls. per capita were sold ; leaving 942 870 galls., or 11.58 galls. per inhabitant. A small portion of this was used in city buildings and for other public purposes, but at least 10 galls. per capita, or 814 000 galls. per day, more than one-third of the total consumption, cannot be accounted for.

In Woonsocket, R. I., all but a very few of the services are metered, and those few are provided with self-closing faucets and have no hose connections. The quantities recorded by the meters and the amounts pumped during the year 1893 are as follows:

	Pumped.	Recorded by meters.	Percentage.
December 1st to March 1st	46 662 321	21 233 500	45.5
March 1st to June 1st	45 763 500	26 995 684	58.9
June 1st to September 1st	61 589 227	31 943 211	51.8
September 1st to December 1st	50 193 139	26 802 669	53.4
Total for year	204 208 187	106 975 064	52.4
Estimated quantity used for street watering, fountains, etc	.....	25 000 000	.....
Quantity used by takers paying schedule rates	.....	10 000 000	.....
Total	204 208 187	141 975 064	69.5

In the calculation of the quantity pumped, an allowance of 5% has been made for the slip of the pumps. Notwithstanding the fact

that more than 80% of the taps are metered, the records show that 62 000 000 galls. per annum, or more than 30% of the quantity pumped, is wasted from the mains and services. These great differences can only be attributed to errors in the quantities registered or to waste and leakage from the street mains and service pipes. It does not seem probable that the loss due to the slip of the pumps and the loss in registration of the meters is more than 20% of the quantity registered by the pumps, in which case there seems to be conclusive evidence that the leakage from the mains is an important factor, and that the use of recording meters will not prevent all waste of water.

The records of the Deacon meters in Boston show conclusively that a very large proportion of the water is wasted. In some of the residential districts the rate of consumption between the hours of 1 to 4 A. M. is from 50 to 100 galls. per capita per day.

Proper Allowance per Capita.—In estimating the quantity to be provided for future years, consideration should be given to the probable legitimate increase in the use of water. Although the greater part of the enormous consumption in some American cities is wasted, there has nevertheless been a great increase in the legitimate consumption during the past 40 years, and without doubt the demands for water will continue to increase in the years to come.

One cause of the increased consumption of water is found in the great increase in the number of water fixtures. Table No. 4 illustrates this growth in Boston.

The proportion of fixtures per capita was nearly twice as large in 1890 as in 1870. Although it is not to be expected that a person will double the use of water because he has twice as many opportunities of drawing it, yet increased use will surely follow from the increase in the number of fixtures.

Another cause of increased consumption is increased pressure in the mains and services. The constant tendency towards increased height of buildings creates a continued demand for increased pressure in the supply mains, and with a large pressure there is more lavish use of water and a greater waste by leakage.

It has been previously stated that 75 galls. per capita is at present an ample allowance for actual uses in a large city. To this must be added an allowance for unavoidable waste and future growth.

The experience of the author leads him to be of the opinion that

it is not practicable to reduce the waste below 15 galls. per capita in our large cities, and that it cannot be maintained at that figure except by the universal use of water meters, aided by Deacon meters or some

TABLE No. 4.

NUMBER OF WATER FIXTURES IN USE AND FIXTURES PER CAPITA, ETC., ON COCHITUATE WORKS, FROM 1870 TO 1892.

Name of fixture.	1870.	Percentage increase.	1880.	Percentage increase.	1890.	Percentage increase.	1892.
Taps	5 893	56.6	9 228	61.7	14 922	12.0	16 706
Sinks	53 010	59.4	84 498	39.7	118 066	6.0	125 151
Bowls	23 961	92.5	46 116	39.8	64 462	6.2	68 448
Bathtubs	8 892	93.8	17 230	85.2	31 914	17.5	37 495
Water-closets	25 050	107.7	52 030	75.4	91 280	12.5	102 687
Urinals	2 447	65.1	4 041	20.7	4 879	—2.6	4 754
Washtubs	9 615	99.1	19 139	126.7	43 389	23.0	53 360
Private hydrants	547	—63.9	197	—81.7	36	—50.0	18
Slop hoppers	723	32.2	956	56.2	1 493	—4.1	1 432
Foot baths	73	90.4	139	101.4	280	—7.1	260
Hydraulic rams	13	—100.0	.....	.....	.....	.....	.....
Totals	130 234	79 3	233 574	58.7	370 721	10.7	410 311
Total population	250 500	31.7	330 000	24.4	410 600	4.8	430 200
Daily average consumption	16 257 700	72.2	28 000 000	21.0	33 871 700	22.0	41 312 400
Fixtures per capita	0.520	36.2	0.708	27.5	0.903	5.6	0.954
Consumption per capita	64.9	31.0	85.0	—2.9	82.5	16.4	96.0
Consumption per fixture	124.8	—3.9	119.9	—23.8	91.4	10.5	101.0

similar device for detecting leaks in the street mains. In cities where water meters are not generally used, the quantity wasted will be from 20 to 100 galls. per capita, as the inspection of mains and house fixtures is more or less rigid.

DISCUSSION.

Mr. FitzGerald. DESMOND FITZGERALD, M. Am. Soc. C. E.—That meters protect the consumer as well as the city against unnecessary expenditures was shown by the speaker's experience. About three years ago, one was placed in his house, and although he did not caution the servants or any of the family against wasting water, there has been a reduction of 30% to 40% in the water bills since that time.

Mr. Rice. L. FREDERICK RICE, M. Am. Soc. C. E.—The speaker's experience had been similar to that of Mr. FitzGerald. Under the old system of rating, his water bills ran from $20 to $25 a year. Eleven years ago he put in a meter and since then no bill has exceeded $6 a year. Reference to the record shows the meter reading to have been 572 on June 2d, 1883, and 41 644 on June 3d, 1895; a total consumption of 41 072 cu. ft., or 307 240 galls. in 12 years. The total cost, at 2 cents per 100 galls., was $61 45, or an average of $5 12 per year.

Mr. Darrach. CHARLES G. DARRACH, M. Am. Soc. C. E.—The figures in the paper concerning the daily consumption of water in Philadelphia, Pa., 150 galls. per capita, are open to question, as the municipal reports for the last few years rate the pumps much higher than their probable capacity. Tests of two of them showed their capacity to be 20% lower than stated in the municipal reports. A great increase in the consumption of water in the city occurred shortly after some large new reservoirs were brought into service, and the speaker believed that there is considerable waste of water through the bottoms and walls of the reservoirs, and not so much along the pipe lines and in the buildings as the recorded total consumption indicates.

Mr. Dunham. H. F. DUNHAM, M. Am. Soc. C. E.—The speaker had found in making tests of entire pipe systems where the mains ranged from 16 to 4 ins. in diameter and a pressure of 50 lbs. per square inch was maintained, that the quantity of water escaping did not exceed 1 gall. per minute per mile of street main. These tests were made when the pipe systems were new and before the introduction of service pipes. They did not include such breaks as would immediately appear on the surface, but did include some leaks from porous castings, faulty joints and imperfectly packed gates that were noticed subsequently. The quantity was determined by plunger displacement. It is one disadvantage of a gravity supply that the quantity of water discharged by a leak of considerable proportions cannot be ascertained so readily or the time of its commencement recorded so clearly as it must be in a well-managed pumping station.

JOHN THOMSON, M. Am. Soc. C. E.— In a general way the speaker was somewhat familiar with the statistical data contained in the paper, but their arrangement to denote the different classes of use and waste not only seemed to be novel, but, in his opinion, is highly valuable. Thus, as shown in Table No. 2, it seemed surprising that portions of the domestic service in Boston, Newton, Fall River and Worcester, and the entire supply of Yonkers, under the relatively free direct service through meters, should be practically no greater than in the instances cited from two districts in London; and this is especially remarkable should it appear that the London districts were partially or wholly under intermittent supply. If the author can answer as to this, and should it be shown, as the speaker assumed, that the London districts were wholly or partially under intermittent supply, then it is about as conclusive evidence as need be desired that the domestic use of water, when paid for according to the quantity drawn, neither leads to restriction nor to lavishness. Mr. Thomson.

Although averages are sometimes deceiving and uncertain, another interesting deduction might be made from this table by dividing the sum of the several per capita consumptions by the twenty-two different conditions of service cited, this giving an average daily use of 24.5 galls. to each person.

As to the quantity of water unaccounted for, as cited on pages 200 and 201, the author's data furnish material for mental exercise, and it is interesting, as well as suggestive, in the instance of Yonkers, to divide the per diem loss of 1 010 000 galls. by the entire number of services, 3 504, thus finding that a leak in each, or a loss in registration by each meter, if all were metered, of 288.7 galls. per day (about 12 galls. per hour) would account for the quantity lost. Now, although the water pressure in Yonkers is higher than that of the average water-works service, ranging from 75 to 125 lbs. to the square inch, it is surprising to find that the 368 650 galls. of water unaccounted for could be delivered in one year through 3 504 thin diaphragms, having comparatively sharp-edged orifices of but 0.03-in. diameter. These figures seem somewhat opposed to some of the speaker's earlier reasoning on this subject;* but the premises have changed somewhat, and in any event this should not stand in the path of a pressing argument. Nevertheless, he was not ready to admit that there is any necessity for attempting to measure water, through meters, by the drop. In Woonsocket, R. I., where 62 000 000 galls., or more than 30% of the quantity pumped, is unaccounted for annually, this amount is equivalent to but about 110 galls. per service daily, less than half the amount at Yonkers, N. Y.

The author has referred in general terms to the different causes of

* "Is the Game Worth the Candle?" *Journal* of the New England Water-Works Association, Vol. VIII, No. 2.

Mr. Thomson. waste, and the speaker, of course, most promptly agreed with the leading conclusion as to "the most efficient means of preventing waste of water," but stated that in this connection it may be of interest to cite one definite instance where the experience was dearly bought.

About three years ago many of the clubs and theaters of New York City were notified by the Department of Public Works to apply meters, among the number being the Engineers' Club. No protest was made and a meter was applied. Several months ago the first bill was rendered, covering about 16 months, showing a consumption of 2 207 250 galls., the amount due being $294 30, or a little over $18 per month. The aforesaid quantity averages a trifle over 3 galls. per minute for the entire period of about 500 days. Every legitimate effort was made to cut at least one figure from the inspector's reading, but in this even the engineers of the department, who are also members of the club, were failures; the bill stood and the club's treasurer begrudgingly paid it. Upon investigation, it was found that a stream had been running constantly, night and day, first to a tank on the roof and thence through the overflow to the sewer. Since that experience the constant flow has been stopped, and the club is now paying about $6 per month instead of $18. A few of such cases, in one system, would seem to indicate that measurement by the drop, while laudable, is not necessary. This query seriously arises: In a case, say, of water scarcity, would not such useless wastes, even if paid for, be bordering upon contributory criminality?

In conclusion, reference was made to the assertion on page 195, which, however, was presumed to be a quotation, referring to the city of Buffalo, that "the people are not in favor of meters," by this conveying the idea that *per contra* the people are in favor of being taxed to increase the present enormous wastage. To this statement, the speaker took direct exception, for he believed that in every such case, if the matter were properly presented to and were directly decided by the citizen taxpayers, they would simply act upon business principles. Would a private water company act otherwise than as here intimated? Then why should a mutual company make it possible for certain of its beneficiaries to take advantage of the others? That such a question should so frequently be up, as if for argument on the merits, is indeed remarkable in a way. And yet there is ground for the opposition. Thus in a report just published by the superintendent of the Richmond, Va., Water-Works, he says: "I do not exaggerate in saying 95% of those paying for water by meter measurement have saved largely in their water bills, and are pleased." But in the conclusion of his report, this significant statement appears: "I have said before, however, that our City Council is opposed to the meter system." And there's the rub betimes; for the beneficent glory of politics, and even the exercise of the engineering function, may be thought by some to be the

better displayed (to the voting populace) by the building of great public works than in the conserving of that they now have. Between politics and engineering conservatism on the one hand, and the exceedingly difficult practical requirements on the other hand, the growth of the meter service has been tedious, a life that has resulted simply as a "survival of the fittest." But, as Tyndall has said, "the plant may be all the hardier for the buffetings it has undergone." Thus, less than a decade ago, the total number of meters made per year by all the manufacturers of the United States did not exceed 15 000, whereas the output for the year 1895 will not be far from 50 000; and the relation of these figures, say as 1.5 : 5, may be taken to fairly represent either the growing necessity for the employment of water meters or the increase of sentiment favorable to their adoption. Mr. Thomson.

J. J. R. Croes, M. Am. Soc. C. E.—In the city of Yonkers, N. Y., to which allusion has been made in the paper, there is a dwelling house occupied by a family of five adults, in which all the rain water that falls upon the roof is collected in a tank in the attic and a cistern above ground, from which water is drawn through a filter of gravel and charcoal. The pipes are connected with the city mains, and when the supply in the tank runs low, the city water is admitted through a meter. Assuming all the rainfall to be collected, the consumption of water in this dwelling for five years past, as derived from the quarterly reading of the meter and the records of rainfall in Yonkers, has been as shown on the following table: Mr. Croes.

Three months ending:	Rainfall.		Metered.	Cubic Feet.
	Inches.	Cubic feet.	Cubic feet.	Total.
August 31st, 1890	13.15	1 460	356	1 816
November 30th, 1890	11.21	1 244	598	1 842
February 28th, 1891	*17.12	1 900	453	3 353
May 31st, 1891	9.81	1 089	703	1 792
August 31st, 1891	6.63	736	1 464	2 200
November 30th, 1891	8.07	896	1 133	2 039
February 29th, 1892	11.39	1 264	765	2 059
May 31st, 1892	8.65	961	1 173	2 134
August 31st, 1892	12.51	1 389	1 168	2 677
November 30th, 1892	9.31	1 033	1 253	2 286
February 28th, 1893	7.50	827	1 179	2 006
May 31st, 1893	*14.12	1 567	1 235	2 802
August 31st, 1893	*12.57	1 396	1 777	3 173
November 30th, 1893	12.37	1 374	1 290	2 664
February 28th, 1894	8.50	944	1 624	2 568
May 31st, 1894	8.47	941	1 502	2 443
August 31st, 1894	4.54	504	1 820	2 324
November 30th, 1894	*22.54	2 502	1 412	3 914
February 28th, 1895	10.22	1 134	1 477	2 611
May 31st, 1895	5.95	661	1 819	2 480
Mean	10.73	1 191	1 210	2 459

Mr. Croes. The mean daily consumption for 1 826 days was 26.94 cu ft., or 202 galls. Omitting the four periods of excessive rainfall, marked in the table with an asterisk, when a portion of the rainfall was wasted by overflow from the cistern, the mean daily consumption for 1 461 days was 24.61 cu. ft. or 184.5 galls., being 36.9 galls. per head of occupants of house. The fixtures in the house are kitchen range and boiler and sink, washtubs, three wash basins, bathtub, and two water-closets, and water is used freely.

Water is not used for lawn sprinkling nor for street or pavement washing, but in summer some water is used for hand sprinkling of flower beds.

Mr. Kuichling. E. KUICHLING, M. Am. Soc. C. E.—The speaker called attention to the great value of the data presented by the author, and was much interested in the recommendation that all of the water drawn from the mains for private consumption should be metered, inasmuch as he had advocated the same measure for many years past, and was thoroughly convinced that it was the only rational method of controlling waste. He also referred to the large variation in consumption of water which occurs in the different seasons of the year in cities where meters are not extensively used, and pointed out that these variations were particularly important in determining, on the one hand, the necessary storage volume that should be available in the reservoirs of gravity works, and on the other hand, the reserve engine capacity needed in pumping works. For such purposes, data of consumption and waste derived from averages are of little value, since the governing figures are necessarily the maximum quantities which occur during comparatively short periods of time, such as a few days, or a week.

In illustration of this point, the speaker referred to the author's estimate that about 6 galls. of water per capita are wasted daily in Boston throughout the entire year, to prevent the freezing of the service pipes in buildings. From the paper, however, it is seen that the difference in consumption between the winter months and the preceding and following months of November and March amounted on the average to 8 225 000 galls. per day, for the period from October 31st, 1892, to April 1st, 1893. Assuming the population at 430 000, as given in Table No. 3, the average waste during the winter months is thus found to be 19.1 galls. per capita per day, or more than three times the average distributed through the entire year. It is to be regretted that the author did not give the figures for the maximum loss from the cause mentioned, as it is very probable that a much larger proportion would have been disclosed. In the case of several small cities, the waste during very cold periods has actually amounted to three or four times the ordinary daily consumption in the spring and fall; and in one city of about 160 000 inhabitants, the consumption for 10 days

during and after a severe blizzard last winter was at more than double the rate prevailing in May and November. Examples of this kind are of importance in connection with other data relating to the use and waste of water from public supplies, as they justify an engineer in making a liberal estimate for either reserve engine power or storage capacity in the design of new works. Mr. Kuichling.

The speaker also called attention to the particular desirability of a general meter system in the case of water-works which are supplied by a gravity conduit, and to the fact that where meters are extensively used the weekly and monthly consumption of water is very much more uniform than in places where meters are not used. Data in support of this fact are not frequently met with, owing to the labor involved in classifying the numerous accounts, and the following statistics which were compiled under the speaker's direction in the water department

STATEMENT SHOWING QUANTITY OF WATER PASSED MONTHLY THROUGH 2 645 METERS DURING THE FISCAL YEAR 1891, AND THROUGH 3 014 METERS DURING THE FISCAL YEAR 1893–94, IN THE CITY OF ROCHESTER, N. Y., FOR THE DIFFERENT USES INDICATED:

Month. Year.	Domestic. Cubic feet.	Commercial Cubic feet.	Manufacturing. Cubic feet.	Total. Cubic feet.
No. of meters	798	1 552	295	2 645
April, 1891	1 671 570	4 047 150	4 055 748	9 774 468
May, "	1 388 309	3 832 223	3 865 201	9 085 733
June, "	1 583 679	3 772 401	3 774 190	9 130 270
July, "	1 427 647	3 746 688	4 245 141	9 419 476
August, "	1 431 356	4 155 907	4 217 450	9 774 713
September, "	1 419 741	4 178 238	3 952 164	9 550 143
October, "	1 481 273	4 142 925	4 088 003	9 712 201
November, "	1 572 703	3 971 598	4 254 038	9 798 339
December, "	1 376 652	3 796 175	3 668 970	8 841 797
Averages, 1891	1 483 659	3 957 031	4 013 434	9 454 127
No. of meters	1 093	1 618	303	3 014
June, 1893	1 654 016	4 632 323	4 197 860	10 484 199
July, "	1 588 105	4 730 581	4 445 094	10 763 780
August, "	1 600 848	4 327 964	4 444 586	10 373 398
December, "	1 471 543	4 161 903	3 891 245	9 524 691
January, "	1 638 259	4 467 307	3 772 614	9 878 180
February "	1 640 456	4 586 693	3 681 974	9 909 123
Averages for three summer months	1 614 323	4 563 623	4 362 513	10 540 459
Averages for three winter months	1 583 419	4 404 301	3 781 944	9 769 664
Averages for twelve months	1 501 966	4 401 526	4 170 755	10 074 247

of the city of Rochester, N. Y., were submitted. The uses of water in that city were divided into three general classes: 1, domestic consumption, embracing all ordinary uses of water in dwellings, apart-

Mr. Kuichling. ment and tenement houses and boarding houses; 2, commercial consumption, embracing uses of water in hotels, restaurants, saloons, commercial buildings with offices and apartments in upper stories, stores, warehouses, livery stables, barber shops, bath houses, and public and charitable institutions; 3, manufacturing consumption, embracing the use of water in breweries, distilleries, paper mills, flouring mills, railroads, fruit-canning works, bakeries, photographers', confectioners', and printing establishments, and in factories and workshops of all kinds.

An examination of the foregoing statement shows a remarkable uniformity in the monthly consumption of water by the three different classes of consumers, the extreme variation throughout being only about 10 per cent. On the other hand, the rates at which the water is actually used throughout the city under the same conditions of pressure differ widely, according to the season; and this fluctuation in consumption must therefore be due to the very large number of unmetered services. In a gravity system, however, which is taxed to its utmost capacity, the supply is practically uniform, and periods of maximum consumption cannot be tolerated without rendering a water famine imminent, but by the general use of meters it is possible to avert such a calamity by rendering the consumption nearly uniform throughout the year, as shown in the statement.

CORRESPONDENCE.

L. J. Le Conte, M. Am. Soc. C. E.—The rate of consumption of water in the large cities of the United States is a perfect anomaly. The paper is a praiseworthy attempt to give more valuable light on the subject, as to its cause and prevention. The marked difference between the rates in small cities as compared with larger ones at once suggests new demands in large cities which hardly exist in the smaller. The writer was much impressed by the classification of the legitimate daily consumption in Boston, namely, 75 galls. per diem per capita, showing for industrial and mechanical uses, 53%; domestic uses, 40%; and public uses, 7 per cent. This at once calls to mind the importance of modern demands for water by hydraulic elevators and water motors. Both of these devices have now become firmly established as public necessities, and their number is rapidly multiplying every year. They have no equal as extravagant consumers of water, as instanced by the fact that special rates have to be applied to such cases, otherwise the water bill presented at the close of each month would assume truly alarming proportions. Mr. Le Conte.

The author's respect for meters as a preventive of waste in general and wilful waste in particular will be shared by every fair-minded person. They have done and are doing an invaluable service toward establishing and maintaining wise economy. The writer thinks it proper to mention one feature about them which, in all fairness, needs to be studied. He has reference to the ratchet meters, which not only record the passage of water through the meter, but also record pulsations in the water main.

His attention was first called to this in 1885, but he neglected to follow it up. In 1889, at the ninth annual convention of the American Water-Works Association, Mr. James P. Donohue, of Davenport, Ia., called attention to unquestionable proof of the fact which came under his own notice. With the general introduction of elevators and their accompanying pulsations or water-hammer, there is a new element of disturbance entering into the problem which will have to be faced and properly solved by the meter manufacturer. A single elevator affects the record of all such meters within many blocks, which happen to be attached to the same main. It is manifestly unfair to have pulsations in the water main charged to the consumer's account.

As to the proper allowance per capita to adopt in designing new water-works there is plenty of room for difference of opinion. The best data available as a guide are supplied by past experience in the same line. Looking back and comparing preliminary estimates with subsequent results there is truly much to make engineers feel humble.

Mr. Le Conte. There is every reason to believe that the legitimate use of water per capita will increase rapidly in the near future, and consequently the writer does not think it prudent to estimate on new water-works for a large city at a lower rate than 200 galls. per diem per capita.

Mr. Herschel. CLEMENS HERSCHEL, M. Am. Soc. C. E.—The principal lesson to be learned from the paper as to a prevention of waste is the old and repeated one—that keeping accounts is at the foundation of an economical conduct of any business, and that to keep accounts in the water-supply business, the water must be measured, that is, metered. To meter large quantities of water, the paper refers to one meter only, the Deacon waste water meter. It is built ordinarily in four sizes only, 4 in., 6 in., 8 in. and 10 in. diameter of the inlet and outlet pipes, and the design of the meter is such that larger sizes are inconvenient in use or impracticable. The horizontal motion of the water in the inlet pipe is turned in a vertical direction, is passed through a circular thin slot, and is then goose-necked back again into a horizontal direction. As a consequence, it is impracticable to use such a meter with an inlet pipe more than 10 ins. in diameter, as it would make the whole apparatus too cumbersome. There is also the necessity of setting a by-pass, for the slot is liable to clog up, and at any and all times the Deacon meter is a great slaughterer of head. As a meter, the apparatus is doing well, when it measures within 5% of the truth. Indeed, its principal function is not to meter, but to indicate relative flows through the pipe under examination, and to draw a diagram of the relative quantities passing.

The use of a meter on mains of any size and on all the principal pipes of the distribution system of a city is, however, perfectly feasible, and would contribute largely to an economy in the use of water. The writer has many times stood by the side of a black and silent 16-in. or 24-in. pipe carrying wash water to a large manufacturing corporation, and wished there were some way of readily procuring a record of what was going on within those hidden and silent confines of the pipe. In the course of years, that wish has been gratified, by the labors of the writer and of others, until now, if the writer were to lay a distribution system, he would certainly place meter tubes in the line of every size pipe laid, and at frequent intervals; so that a strict account could be kept at any time of the actual distribution of water within districts of any desired extent or limitation.

The meter tubes referred to are those of the Venturi meter, described* in the *Transactions* of this Society, and now forming an article of commerce, both in the United States and abroad. The meter-tubes of this meter are of a length equal to about 10 times their diameter, and their cost is about three times that of an equal length of plain pipe. No by-pass is needed, or has ever been used, to the writer's

* Vol. XVII, 1888, page 228.

knowledge; the head lost is a trifle, not so much as is destroyed by an ordinary check valve. Any size pipe may be fitted with these tubes, from ¼ in. to 240 in. or more; the register, to register total cubic feet passed, or to make a diagram of the rate of flow, or to perform both of these functions simultaneously, is applicable to any size of meter tube and meters the water passing it within ½% of the truth, for any single record reading. This register could readily be used as a portable register, and be connected at will with the selected meter tube by two little pipes conveying pressure, not a stream of water, from the tube to the register. For city street use or for waste water inspection, this would be the proper way to set the whole apparatus; the meter tube set permanently in the line of the street pipe and constructed so as to meter water flowing in either direction whenever there is any possibility of reversed currents in the pipes; the register set temporarily at any convenient place within 100 or 1 000 ft. of the tube, and connected with it as described. Where used simply as a meter and to register total quantities passed, the register is set permanently in some convenient place, not too far from the tube. Many such meters have been doing daily service for the past three years at home and abroad. Mr. Herschel.

It is time that civil engineers recognize in their general practice that a pipe conveying water need no longer be the dead, silent, secretive factor it usually is, or as it has above been described from old-time recollections of the writer's practice, but that the introduction of a Venturi meter tube in the line of a pipe can cause it to speak with accuracy and precision respecting the flow of water within it at any desired time. Were pipes generally thus made to reveal at will the flow of the water within them, and were systematic records of this flow kept in cities, the waste of water, now furthered by the absence of all information on this subject, would materially diminish. Certain it is that the unseemly record of broken 4-in. and 6-in. pipes wasting 1 000 galls. per hour for years without detection would then no longer be possible.

Robert Cartwright, M. Am. Soc. C. E.—The writer read the paper with much interest, and the statistics therein given are a valuable contribution to the profession. He fully agrees with the author in his conclusions, that mainly by the metering of all services can the lavish waste of water be controlled. The writer's connection for a period of some 30 years with the manufacture and distribution of illuminating gas has convinced him of the necessity of placing some restraint upon the prodigality of consumption so prone to the American people, and which has given rise to the trite saying that "any other nation in the world could live well upon the waste of the people of the United States." Mr. Cartwright

No doubt much of the waste of water results from the parsimony of landlords in saving expense by putting in cheap plumbing and materials, and the running of water to prevent freezing, as shown by the paper.

Mr. Cartwright

The periodical inspection of every service the writer regards as very essential for the detection of waste, as no doubt many instances happen without regard to the material or the quality of the plumbing and in unlooked-for ways, as in the facts herein given occurring in the writer's experience. In Rochester, N. Y., all sprinkling of lawns has been prohibited for some years, owing to the water not being available, by reason of lack of facility to convey the water. Last fall a new supply main was completed, and citizens now have the opportunity of using all the water they wish, provided they purchase such meters to register the consumption as are tested and approved by the water officials. Water is supplied at the rate of 14 cents per 1 000 galls. The writer availed himself of this provision, and after due consideration and investigation, naturally selected a disk meter as being the most accurate, simple and economical for his uses. The meter was tested in his presence by the water officials, and found absolutely correct at both maximum and minimum flow.

The writer's residence was built in the fall and winter of 1892, and was equipped with the best plumbing to be had, both in workmanship and materials. The meter was set and put in operation Saturday, May 5th, 1895, at noon. At 10 P. M. that night, the registry was somewhat surprising to the writer, as it indicated a remarkable industry on the part of the meter. Absence from home prevented another inspection until 6 o'clock P. M. on Monday, May 7th. Surprise gave place to disgust when the register showed 476 cu. ft. as having passed in about two and one-half days, and a remedy was sought. The water is supplied by an independent pipe from the street main to a tank holding about 100 galls. in the attic, and fitted with a valve controlled by ball float. An overflow is provided from the tank to the sewer in case of any accident to the float valve. From the bottom of the tank, connection is made with the house system of piping, with a cock just outside the tank. In case of a lack of pressure in the street mains to raise the water to the tank, a by-pass is placed in the cellar, between the independent riser and the house system and fitted with a ¾-in. globe valve. With this arrangement, an approximately constant and reduced head is maintained on the house system, when the by-pass is shut.

An examination of the float valve resulted in putting a new seat on it, although the leak was trifling. When the water was turned on after making the repairs, the tank began filling, although no water was passing the float valve. This proved that water was coming *via* the by-pass, notwithstanding the valve was closed. The cap of the ¾-in. by-pass valve was removed, and showed that the seat had never been put in, and it had evidently been sent from the factory in that condition. Here, then, was the solution of the question. A new seat was put in, and the writer's faith in meters was restored. For over two years the water had been running directly to the sewer, through this

defective valve, and enough had been wasted to have supplied several families in the interim. A night inspection and application of the waterphone would have detected this loss. This is only one instance where the application of a meter divulged the waste, and undoubtedly there are many just such cases existing. The writer views the metering of water as having a moral influence upon the majority of consumers, since when a check is placed upon the consumption of water in the shape of a tell-tale mechanical watchman, the party paying the bill is very apt to count the cost of carelessness or indiscretion, and will prevent a waste that otherwise would not be considered. In fact, the writer believes that a meter case without any works inside, applied to a service, would serve an admirable purpose in reducing waste in many instances. Mr. Cartwright

For years the writer has been convinced that the metering of every water service (the meter to be furnished free to the consumer, as is the case in illuminating gas supply) would cost less in the outlay for meters than the outlay for new conduits, reservoirs and pumps, with their maintenance, to say nothing of the waste of potable water.

A community will vote thousands of dollars to extend its water supply more readily than it will expend hundreds in supplying meters which will curtail and control the waste. No private individual would long carry on the business of pumping and supplying water when no check is placed upon the consumption, and yet municipalities almost universally adopt that plan.

The daily use of water in the writer's family is as follows: The family consists of six adults; the house is fitted with bath, two closets, two wash basins, three stationary wash tubs, beside kitchen and pantry supply, and embraces 19 faucets. The minimum rate of water used is 16 cu. ft. per day, the maximum is 54 cu. ft.; the average for about three months is 34.83 cu. ft. per day, Sundays being the lowest and Mondays (wash day) the highest. This shows an average use of 261 galls. per day for the family, and an average per capita per day of 43.5 galls.

According to the author's statistics, Philadelphia, Pa., uses 150 galls. and Buffalo uses 252 galls. per capita per day, and the question forcibly presents itself, What shall the engineer assume as a reasonable per capita estimate, when planning a water supply? To the writer's mind, the meter offers the only solution of the question, and when every water service is metered, 75 galls. per capita per day would seem an ample provision for a supply.

Daniel W. Mead, M. Am. Soc. C. E.—The increase in the per capita consumption of water in most of the cities of the United States is now very rapid and is fast becoming a serious matter. In many cases, it is almost or quite impossible financially for the cities to enlarge their works fast enough to keep up with the extravagant use of Mr. Mead.

Mr. Mead. water. In the majority of cases, a large amount of the water used is unquestionably wasted, and in many instances much better results would be obtained if the city officials confined their attention to the restriction of waste rather than the enlargement of the works.

In the fall of 1894, the writer was called to examine and report on the enlargement of the water-works system of the city of Rock Island, Ill. This is a city of perhaps 18 000 inhabitants, and draws its supply from the Mississippi River. While having a reasonable number of manufacturing establishments, it has no great manufacturing interests which would consume large quantities of water, and there is no apparent reason for the excessive consumption of water which is taking place.

The following table gives the average daily consumption per capita for each month of each year since 1889, and exhibits the excessive consumption in that city in detail.

TABLE SHOWING CONSUMPTION OF WATER AT ROCK ISLAND, ILL.

YEAR.	Reported and estimated population.	Mean daily consumption. Gallons.	Average daily consumption per capita.* Gallons.	AVERAGE DAILY CONSUMPTION PER CAPITA FOR EACH MONTH.											
				January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.
1850.....	1 711														
1860.....	5 130														
1870.....	7 890														
1880.....	11 659														
1889.....	13 595	1 935 647	140	....	....	....	157	152	154	153	152	158	153	143	135
1890.....	13 890	1 702 395	121	121	120	109	97	109	116	147	155	146	145	114	116
1891.....	14 180	2 020 372	142	110	112	104	106	133	156	172	176	168	138	144	132
1892.....	14 465	2 105 836	144	134	128	117	118	122	137	150	158	156	150	149	145
1893.....	14 745	2 174 031	146	156	158	145	144	158	158	159	170	162	143	140	129
1894.....	15 020	2 528 509	167†	133	138	136	145	156	184	185	172	....	....	....	....

* Averages from April to April.

† Average from April to September.

To ascertain the rate of consumption for different portions of the 24 hours, observations were taken for two days in September and for the three days, November 30th, December 1st and 2d, 1894. From the half-hourly readings of the pump counters, Fig. 2 was plotted. It will be noticed in the part of the diagram for September, that the minimum rate is almost 73% of the average, and about 56% of the maximum rate, the minimum rate at night being about 1 400 galls. per minute. In November and December the average minimum rate is about 90% of the average rate and about 72% of the maximum rate. While the maximum rate of 2 500 galls. per minute in September has fallen in November and December to 1 600 galls. per minute, a decrease of 900 galls. per minute, the minimum rate at night has decreased only

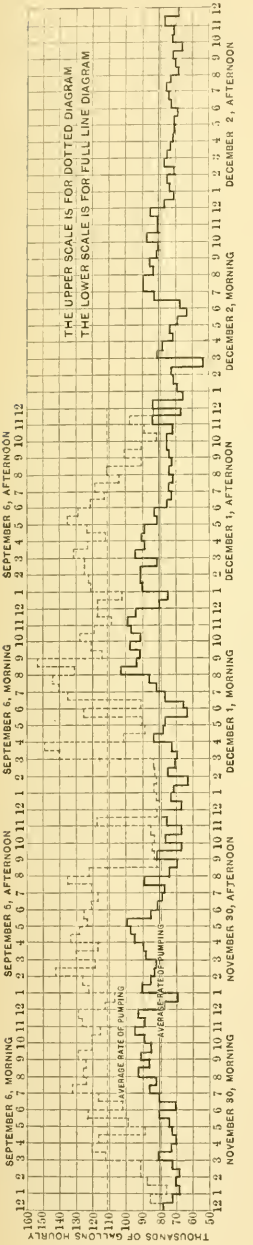

THE UPPER SCALE IS FOR DOTTED DIAGRAM
THE LOWER SCALE IS FOR FULL LINE DIAGRAM
SEPTEMBER 6, MORNING
SEPTEMBER 6, AFTERNOON
SEPTEMBER 6, MORNING
SEPTEMBER 6, AFTERNOON
NOVEMBER 30, MORNING
NOVEMBER 30, AFTERNOON
DECEMBER 2, MORNING
DECEMBER 2, AFTERNOON
DECEMBER 1, MORNING
DECEMBER 1, AFTERNOON
AVERAGE RATE OF PUMPING
AVERAGE RATE OF PUMPING
THOUSANDS OF GALLONS HOURLY

FIG. 2.

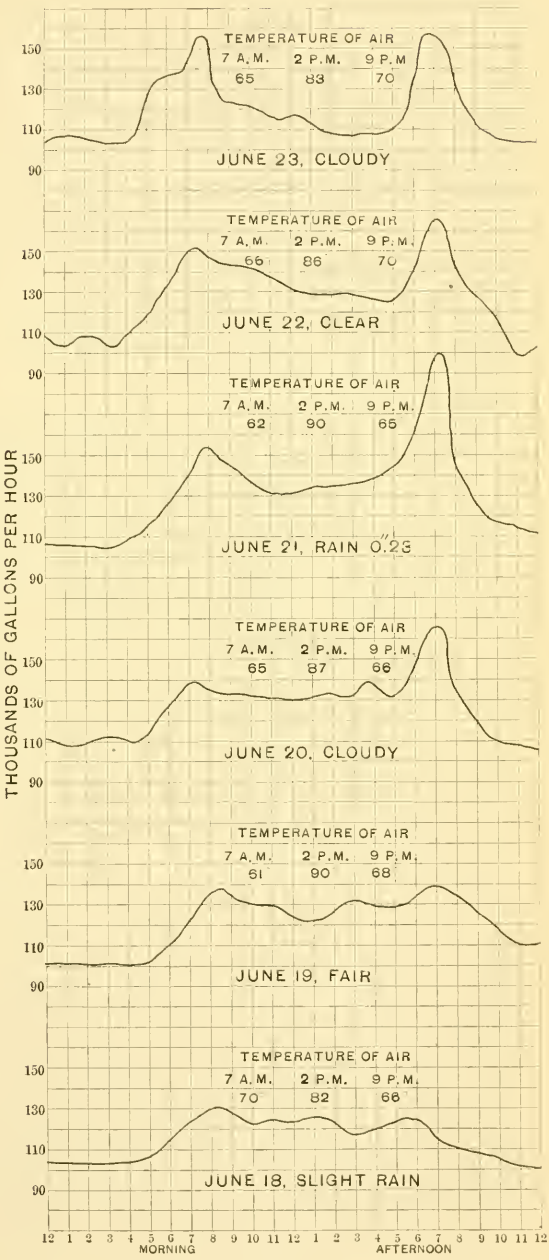

TEMPERATURE OF AIR
7 A.M. 2 P.M. 9 P.M.
65 83 70
JUNE 23, CLOUDY
TEMPERATURE OF AIR
7 A.M. 2 P.M. 9 P.M.
66 86 70
JUNE 22, CLEAR
TEMPERATURE OF AIR
7 A.M. 2 P.M. 9 P.M.
62 90 65
JUNE 21, RAIN 0.″23
TEMPERATURE OF AIR
7 A.M. 2 P.M. 9 P.M.
65 87 66
JUNE 20, CLOUDY
TEMPERATURE OF AIR
7 A.M. 2 P.M. 9 P.M.
51 90 68
JUNE 19, FAIR
TEMPERATURE OF AIR
7 A.M. 2 P.M. 9 P.M.
70 82 66
JUNE 18, SLIGHT RAIN
THOUSANDS OF GALLONS PER HOUR
MORNING
AFTERNOON

FIG. 3.

Mr. Mead. about 80 galls. per minute. There is evidence in the diagram of an excessive waste of water, which, if remedied, would have a marked influence on the relative capacity of the plant. Some of the smaller irregularities in the diagram can probably be accounted for by probable irregularities in the reading of the counter.

Fig. 3 shows the mean curve of the rate of consumption of water at Rockford, Ill., for six days in June, 1895. Rockford is a city of about 30 000 inhabitants, and the average consumption of water is about 90 galls. per capita, considerably less than that of Rock Island. The large consumption of water during the night, when the consumption should be relatively small, would, however, seem to indicate an excessive waste of water. The diagram shows the large consumption during the sprinkling hours and the effect of a higher temperature on the use of water.

Mr. Hazen. ALLEN HAZEN, Esq.—The statistics on page 219 in regard to the consumption of water in 30 cities in Germany and German Switzerland have been taken from the excellent statistical report of the German Society of Gas and Water Experts for the year 1893. This report contains statistical information with regard to 75 water-works, from which the 30 were selected, either because the statistics were more complete than for the others, or because the cities were larger and have greater interest. The quantities are stated in the original report in cubic meters annually for the entire city, and have been reduced to gallons per inhabitant daily by the use of a slide rule. They are thus only approximately exact, although it is believed there are no large errors, because the totals have been checked with the sums of the various parts in each case after reduction.

In addition to the data given in the table, this report contains detailed information in regard to the quantities of water used for the various public purposes in many of the cities, although the quantities of water used for different purposes are often not stated in full, particularly where the total quantity for public purposes is but small.

The water-works themselves require a quantity of water ranging from nothing to over 6 galls., and averaging for 16 cities 0.54 gall. per inhabitant daily. Street sprinkling requires from 0.01 gall. to 1.10 galls., averaging 0.34 gall. for 28 cities. A further quantity of water, ranging from nothing to 1.35 galls., was used for watering parks, and averaged 0.23 gall. for 22 cities. Ornamental fountains required from 0.01 gall. to 3 galls., and in one case 6 galls., averaging 0.66 gall. for 26 cities. Drinking fountains required from 0.01 to 5.23 galls., averaging 0.95 gall. for 21 cities. Flushing and washing gutters required from 0.01 to 0.49 gall., averaging for 7 cities 0.08 gall., and sewer flushing required from 0.01 to 1.52 galls., averaging 0.42 gall. for 17 cities. Public closets required from 0.02 to 1.69 galls., averaging 0.53 gall. for 26 cities. From 0.001 to 0.63 gall. was required for

extinguishing fires, averaging 0.07 gall. for 18 cities, but in only two cases was the quantity above 0.025 gall. Nineteen cities report water used for special purposes not specified, ranging from 0.001 gall. to 4.43 galls., averaging 0.82 gall. The sum of these averages is 4.64 galls., but owing to the incompleteness of the returns, and to the fact that probably many of the cities use no water for some of the purposes, and as zeroes are not included in the average, the result so calculated is considerably higher than the 3.61 galls., which is the average quantity for all public purposes shown in the table. Mr. Hazen.

USE OF WATER IN SOME GERMAN CITIES.—GALLONS PER CAPITA DAILY.

Place.	Population.	Consumption.	Public purposes.	Private. Metered.	Private. Estimated.	Not accounted for.
Altona	156 500	26.70	.....	.....	.....	.....
Barmen	118 500	33.59	1.33	11.30	17.60	3.36
Basel	74 500	34.78	7.47	19.80	0.96	6.55
Berlin	1,606 424	16.37	1.63	14.45	.....	0.29
Bonn	52 000	24.94	0.79	11.40	12.50	0.25
Breslau	335 000	21.71	6.01	15.70	.....	.....
Chemnitz	139 374	11.50	3.35	8.15	.....	.....
Cologne	255 000	45.22	8.92	5.70	30.60	.....
Crefeld	105 712	18.52	0.61	4.71	13.20	.....
Danzig	107 085	26.70	.....	11.30		.....
Dresden	280 200	21.54	1.80	12.39	7.30	0.05
Dusseldorf	155 900	22.10	1.44	10.80	7.65	2.21
Elberfeld	137 000	29.92	7.16	22.75	0.01	.....
Frankfort	186 000	36.26	2.56	5.40	28.30	.....
Freiburg, B.	48 200	41.46	7.00	12.46	22.00	.....
Hamburg	583 700	58.00	2.87	.....	.....	.....
Hanover	189 976	18.40	1.15	4.26	13.00	.....
Halle	120 000	21.94	1.18	10.06	10.70	.....
Karlsruhe	74 200	28.14	8.64	6.20	13.30	.....
Kiel	72 000	20.18	2.22	2.09	15.46	0.41
Königsberg	162 000	16.87	1.21	2.06	13.60	.....
Magdeburg	198 000	25.24	2.22	20.60	0.01	2.43
Munich	298 000	34.00	1.00	31.30	1.70	.....
Nuremburg	145 000	17.41	4.03	8.10	3.35	1.93
Posen	70 000	13.33	2.01	9.22	0.06	2.04
Stettin	118 000	31.54	2.37	3.67	25.50	.....
Stuttgart	139 200	21.34	5.60	6.75	7.90	1.04
Wiesbaden	66 000	20.74	6.14	12.70	0.10	1.80
Wurzburg	61 032	35.50	.....	27.00	.....	.....
Zurich	96 650	56.71	6.76	19.45	30.50	.....
Average	.....	27.69	3.61	.....	.....	.....

No separate accounts are kept of the quantities of water used for manufacturing and domestic purposes, both being classed as water sold and paid for. Manufacturing is extensively carried on in many of the cities, but the writer judges that a much larger proportion of establishments have their own independent and cheaper supplies than is the case in America. This is particularly the case in Berlin, where even boilers in houses are commonly fed with well water, which is everywhere easily obtained, but is not allowed to be used for domestic purposes.

The Berlin water-works afford an unusually interesting example of

Mr. Hazen. accounting for all of the water pumped. The following table has been freely translated and slightly condensed from the annual report for 1892-93, and indicates only 2½% of water lost through leakage, under-registration of meters and other causes:

WATER USED IN BERLIN, 1892–93.

	Cubic meters.	Percentage.	Gallons per head daily.
I.—For water-works purposes, feed water for boilers and testing meters	296 533	0.741	0.131
II.—Supplied without payment for public purposes.			
A.—METERED.			
Sprinkling public parks and gardens	264 986	0.662	0.118
Cleaning public monuments	261	0.001	0.000
Fourteen public ornamental fountains	702 079	1.754	0.311
Used by seven public closets and urinals	9 754	0.024	0.004
Used for military telegraph station	200	0.000	0.000
Used for flushing sewers	1 306 950	3 263	0.578
B.—ESTIMATED IN VARIOUS WAYS.			
Flushing gutters	82 056	0.205	0.036
Extinguishing fires	4 382	0.011	0.002
Street sprinkling	1 115 063	2.785	0.495
Watering trees in public streets	23 927	0.060	0.011
Used by 140 public closets and urinals	745 620	1.863	0.330
At the "Urania" exhibition	7 600	0.019	0.003
To this add loss through leakage, emptying and filling mains, under-registration of meters and some special uses	1 028 225	2.568	0.455
	5 290 603	13.215	2.343
III.—Used and paid for by water-takers by meter measurement	34 448 786	86.044	15.276
The water is thus divided as follows:			
I.—Water-works purposes	296 533	0.741	0.131
II.—Public purposes without payment and loss	5 290 603	13.215	2.343
III.—Paid for by water-takers	34 448 786	86.044	15.276
	40 035 922	100.000	17.750

It should be stated, however, that in calculating the quantity of water supplied, an allowance of 10% is made for the slip of the pumps.

In regard to the meters it might be said that in general all the water used in an apartment house is measured at one place, and paid for by the owner of the building, but on large services two meters, a large and a small one, are used, the larger meter registering the flow when the velocity exceeds the capacity of the smaller meter and the smaller meter registering when the flow is too slow to afford accurate results with the larger meter. The water is automatically taken through either one or the other of the meters, according to the velocity passing, and the sum of the quantities registered is the total quantity passed.

Dexter Brackett, M. Am. Soc. C. E.—In reply to Mr. Thomson's inquiry, the author will say that the London districts were under constant supply, and the conditions were practically the same as in the case of Newton, Fall River and Worcester. The experience of the New York Engineers' Club is almost identical with that of hundreds of water-takers. Unseen leaks are allowed to continue for years, until the application of a meter and the presentation of a water bill proves their existence. Mr. Brackett.

Mr. Kuichling calls attention to the fact that the quantity used in the different months is much more uniform when the water is supplied through meters. This is without doubt due to the fact that the extravagant use of water during the summer, and the waste to prevent freezing of service pipes in the winter, are both largely prevented by the application of meters. The property-owner finds a way to protect his service pipe from freezing when he is obliged to pay for the water used.

While the author agrees with Mr. Le Conte that there will be in the future a legitimate increase in the per capita use of water, he does not believe that a supply of 200 galls. per head should be provided for our large cities. The use of water for hydraulic elevators and motors should not be encouraged, except where the supply is unlimited and near at hand.

There is no doubt but that the Deacon meter in sizes larger than 12 ins. would be cumbersome, but for general use in waste detection sizes larger than 6 ins. are rarely, if ever, wanted. The 6-in. meter is the only size found necessary in Boston, and in England the 4-in. meter is very generally used.

An experience of 15 years has never shown any necessity for placing the meter on a by-pass when used in connection with a pipe of the same size, on account of the danger that the waterway may become clogged, or because of the head lost. The custom in England is to place the meters directly on the line of the pipe, but the author has found it more convenient to place them under the sidewalk on account of convenience of access, and for the further reason that the 6-in. meters are often used in connection with 12-in. mains.

As the area of the waterway through the meter is equal to or larger than that of the pipe with which it is connected, it is difficult to understand how any substances found in drinking water could clog the meter, and, as the velocity through the meter seldom exceeds 5 ft. per second, the head lost is inappreciable.

Mr. Herschel claims that in the case of the Venturi meter no by-pass is needed, and that the head lost is a trifle. In the Venturi meter, as ordinarily made, the throat of the meter has a diameter one-third that of the pipe, with one-ninth of the area, and the velocity of the water is consequently increased nine times. It would seem prob-

Mr. Brackett. able that a 6-in. pipe contracted to an opening 2 ins. in diameter would be more liable to cause loss of head in a pipe system than the full area waterway of the Deacon meter.

Under some conditions, in large mains for example, where the changes in velocity are not great, the Venturi meter, without doubt, will give good results, but when it is remembered that the head varies as the square of the velocity, and that the quantities measured by the 6-in. Deacon meter vary from 1 000 to 15 000 galls. per hour, it is evident either that the head lost at the higher velocity will be excessive, or that the head at the lower velocity will be difficult to record.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

759.

Vol. XXXIV.—September, 1895.

THE SEPARATE SEWER SYSTEM WITHOUT AUTOMATIC FLUSH TANKS.

By F. S. ODELL, M. Am. Soc. C. E.

READ AT THE ANNUAL CONVENTION, JUNE, 1895.

WITH DISCUSSION.

In the *Transactions* of this Society, Vol. X, page 29, the statement is made: "The flush tanks which make the separate system practicable are placed at the head of each branch sewer."

So far as the author is aware, this statement remains unchallenged in the literature on this subject up to the present time. It was held that unless a system of sewers excluding storm water was flushed by the artificial introduction of water at frequent intervals, stoppages would most assuredly occur, and to such an extent as to greatly impair, if not entirely vitiate, the usefulness of such a system. The supposed necessity for frequent flushing speedily brought into existence a number of excellent devices for automatically discharging the contents of storage tanks or cisterns when filled to a certain level, and thus it is possible to regulate the periods of flushing by limiting the streams by which the tanks are fed. These automatic flush tanks have been adopted almost universally as an integral part of the separate sewer system, and doubtless engineers still consider them indispensable adjuncts.

William E. Worthen, Past President Am. Soc. C. E., entertained serious misgivings concerning the virtue of frequent flushings of sewers with such small quantities of water as the capacity of the ordinary flush tank. The inadequacy of the water supply to furnish even the small additional quantity of water for operating flush tanks on the sewer system at Mount Vernon, N. Y., in the years 1888 and 1889, gave a convenient excuse to abandon the theoretical necessity of automatic flushing, and to test in a practical manner, on a sufficiently extensive scale, how a separate system of sewers would behave when flush tanks were entirely omitted. The system of sewers, about 20 miles, was therefore constructed without flush tanks. Manholes were located at all dead ends, and at intervals of about 400 ft. along the lines of sewers; the minimum size of pipe sewer was fixed at 8 ins. diameter, increasing in size in the district trunk sewers to 20 ins.; they were laid in the usual manner, true to line and grade, all changes in direction being made at manholes.

By an ordinance of the Health Department, all surface and storm water was excluded from the sewers, except that roof water from one or two buildings at each summit or dead end was allowed to enter for flushing purposes. The general inspector also deemed it wise to use additional precaution against stoppages, and therefore ordered frequent flushings from fire hydrants, introducing the water through 2½-in. hose into manholes along the line of sewers.

This was the condition of things in 1892, when, as Commissioner of Public Works under a new city charter, the author assumed charge of the sewer system. It was not without considerable trepidation and visions of serious trouble from clogged sewers, flooded basements and cellars, that the work of investigating the condition of the sewers was begun. Frequent connections with the sewers at that time materially aided in this examination, and disclosed a very wholesome and satisfactory condition indeed. No deposits were found, no gases could be detected, nor any objectionable odor.

It was not long, however, before an unexpected difficulty manifested itself. A severe rainstorm occurred, and a deep basement in the lower part of the city became flooded to a depth of 5 or 6 ins. An examination of the sewer in that vicinity showed an abnormal quantity of water standing 3 or 4 ft. in the nearest manhole. Continued observation of this locality at each recurring rainstorm established the fact

that too much storm water was finding its way into the sewers, and while not a little entered through the perforated covers of the manholes, it became evident that it came also from other sources. Up to this time no jurisdiction had been exercised by the municipality over the plumbing in buildings connected with the sewers, and no inspection made of any portion of the drainage system of dwellings or other buildings, save that portion of the drain within the limits of the street.

It became necessary to exclude storm water from the sewers. An ordinance was passed by the Common Council revoking all permits for connecting rain-water leaders with the sewers, and an inspector was appointed to enforce the ordinance. A house-to-house inspection was made of all premises connected with the sewers, and storm-water connections cut off wherever found. About this time the State law requiring plumbing inspection in all cities went into effect, so that all connections since made are subject to inspection and storm-water connections precluded. Complaints of back water from sewers have ceased, and, so far as known, storm water is now entirely excluded from the sewers.

During the year 1893 periodical flushing was continued, but frequent observations seemed to warrant the lengthening of the intervals between flushings, until in 1894 the intervals were extended to six months, and this period is now regularly maintained. The spring and fall seasons are selected for this work, there being an abundance of water available then. It is considered advisable to continue semi-annual flushing, although the necessity for it does not appear. In all new work the author is having direct connections made between the water mains and dead ends of sewers by the use of a 4-in. pipe and gate; with this the work of flushing is done very quickly and effectually.

It may be interesting to note here that during the three years covered by the author's observation and control of this sewer system, six stoppages have occurred. Two were occasioned by excavations carried below the sewers, the pipes being broken from settlement of the loose material; two more were caused by bedding and whole garments improperly permitted to enter the sewers; one stoppage was the result of coal tar escaping from the gas works, and last but not least, the same mischief was the work of the roots of a willow tree. Singularly enough, this last was the only stoppage in the lateral sewers; all the others occurred in district trunk sewers, with a constant flow of sew-

age, and so far removed from the dead ends as to be scarcely susceptible to the influence of flush tanks, had they been in use.

It is not the purpose of this paper to advocate the omission of flush tanks from all separate sewer systems, but to show they are not an essential feature; and when, from any sufficient reason, such as an insufficient or costly supply of water, they may be entirely dispensed with.

DISCUSSION.

Mr. Darrach. C. G. DARRACH, M. Am. Soc. C. E.—In the village of Ridley Park, Pa., where the speaker resided, there has been a separate system of sewerage for 16 years, which has grown gradually to a length of approximately 5 miles. The drains from the houses to the sewers are 3 to 4 ins. in diameter, and the sewers are 4, 6, 8 and 12 ins. in diameter. During the entire period of the speaker's residence in the town there had been no clogging of the sewers, although the only flushing water admitted to them is a small amount of rain water at the ends.

Mr. Hering. RUDOLPH HERING, M. Am. Soc. C. E.—The speaker was pleased to see the paper expressed the same views on automatic flushing which he had held for 16 years, but he was surprised to hear the author state it had been the general opinion of engineers that flush tanks were necessary for the proper operation of the separate system. In 1880 he inspected the existing separate sewerage systems in England, constructed in accordance with the principles since largely followed in this country, and they were not flushed by automatic flush tanks. He examined them very carefully by descending into the manholes, looking up the pipes and otherwise at all places where they were accessible, and found they were kept in very good condition without the use of such automatic flush tanks. He did not wish to be understood as opposed to these appliances in themselves; on the contrary, he had frequently recommended them. Under certain conditions they are very useful, economical and effective, particularly where the cost of water is low enough to permit the use of especially large tanks. He had found the automatic tanks of more advantage on steep than flat grades, the advantage being that the bulk of the water passes through the pipe somewhat like a piston, thus cleaning it more effectually. If the grades are light, discharges of 200 to 300 galls. generally lose their flushing power after passing a few hundred feet through the pipe, and 500 ft. from the tank he had sometimes been unable to detect any difence in the flow due to the operation of such an apparatus. While their use is to be recommended in many places, they certainly do not form an essential part of the separate system of sewerage.

CORRESPONDENCE.

Mr. Kiersted. WYNKOOP KIERSTED, M. Am. Soc. C. E.—The author has raised an interesting point regarding the relation of automatic flush tanks to the separate system of sewers in assuming that engineers generally considered them indispensable adjuncts to separate sewerage. The quotation contained in the first paragraph of the paper refers specifically to the Memphis sewer system, and, like several other claims for that sewer system which were made at the time of its construction, has never been generally accepted as correct so far as the writer is aware. If the quoted statement has remained unchallenged in current literature, it is, perhaps, because the flushing of separate sewers, even though at times unnecessary, and the attachment to the sewer system of apparatus intended for that purpose do not in any particular affect prejudicially the operation of the sewers.

As the writer understands the question of flushing, the prevailing opinion among engineers is that while the flushing of sewers may be satisfactorily accomplished, so far as the immediate effect upon the sewers is concerned, by confining flushing water from a hydrant in a manhole at the summit of a lateral until full, then suddenly releasing it, or by means of a portable flush tank; still, in cases where frequent flushing is desired, the automatic flush tank affords an economical and convenient means of accomplishing the desired result in the upper portion of light grade lateral sewers, which seldom, if ever, flow more than a mere driblet of sewage containing solid matter but partially disintegrated. The question of flushing and the means to that end are associated largely with questions bearing upon the maintenance and operation of a sewer system after construction. In determining whether flushing is or is not necessary, the sewer gradient becomes a controlling factor. If flushing be thought necessary, considerations of economy in maintenance and operation usually decide, or should decide, whether automatic flush tanks or other flushing devices be used. This observation may apply, not only to the separate system of sewers, but also in a greater or less degree to the combined system.

The tendency of undisintegrated solids to deposit in the upper portion of a lateral sewer, and to thereby obstruct it, affects the flow of a sewer of heavy gradient much less than it does one of light gradient, as usually the accumulated head of sewage behind the obstruction in the former case would be sufficient to remove the obstruction, while in the latter case assistance by flushing or otherwise may occasionally be found necessary. In this connection it would be interesting to know the grades of the Mount Vernon sewer system.

So far as the writer's experience and observation affords a basis for

Mr. Kiersted. an opinion, he believes that the efficient flushing effect of the automatic flush tank is confined to a very limited length of sewer and that in many instances it may be dispensed with without prejudicially affecting the sanitary condition of the lateral sewer. Occasional flushing in the manner which the author describes will, in all probability, be found in many places other than Mount Vernon to meet all the demands for preserving cleanliness in the sewers of the separate or combined system, and often even more efficacious than the automatic flush tank for that purpose. In one sewer system which the writer designed he recommended an automatic flush only on laterals of less than 0.5% grade. The system has now been in operation five years with a record of but few stoppages. The minimum size of pipe is 8 ins., and all changes of direction are in the manholes.

Mr. Allen. KENNETH ALLEN, M. Am. Soc. C. E.—The extent to which automatic flush tanks may be omitted from sewers of the separate system is well worth consideration on the grounds of both sanitation and economy. It is a question of the relative amount of sludge to be transported and the available grade. The writer thinks, from the wording of the paper, that the erroneous impression might be received that the accredited omission of flush tanks is a novel practice, whereas, since the suggestion of a separation of the house sewage in England in 1842, by Mr. Edwin Chadwick, and its introduction soon after, it was a good many years before what is known as the automatic flush tank was introduced into sewerage works. In many instances the rain water from leaders was depended upon; in others, as in Liverpool to-day, portable tanks are emptied periodically into manholes, while in others all special means of flushing have been omitted.

The writer is a thorough believer in flushing sewers of the separate system by some method under ordinary conditions, but the experience at Mount Vernon has indicated that with good grades and well-built sewers, flush tanks are not as essential as is commonly supposed. The effect of the flush, it is true, does not extend far below the tank. To ascertain this decrease in the flow, the writer made a few observations of the discharges from Rhoads-Williams tanks put in by him at Fort Supply, I. T., some years ago. The results, which are given in the following tables, were for clean and empty sewers, receiving no other water. The pipes were in 2-ft. lengths and were laid to line and grade with exceptional care. As gaskets were not used, the work was new and the soil a perfectly dry open sand, it is probable that some slight loss from leakage occurred, but the amount was not ascertained. The results given are of course too meager in number and the local conditions were too unusual to afford a basis of calculation for other work, and are given here as there seems to be scarcely any printed data on the subject. They show clearly the limited value of the flush when there is not a good flow of sewage or a coincidence in the times of discharge.

Flush Tank "H."

Capacity, 360 galls. Time to discharge, 46 seconds.

Reach.	Size of sewer.	Distance.	Grade.	Average velocity, front of discharge, Feet per second.	Duration of maximum discharge.	Maximum depth on invert (approximate).
Tank to Manhole 1.....	6 ins.	558 ft.	1.09%	5.03	1 min. 57 sec.	3½ ins.
Manhole 1 to Manhole 2.	6 "	260 "	1.79%	4.56	3 " 18 "	3 "
" 2 to " 3.	6 "	260 "	0.50%	3.17	2 " 30 "	3 "

Flush Tank "K."

Capacity, 360 galls. Time to discharge, 54 seconds.

Reach.	Size of sewer.	Distance.	Grade.	Average velocity, front of discharge. Feet per second.	Duration of maximum discharge.	Maximum depth on invert (approximate).
Tank to Manhole 4.....	6 ins.	680 ft.	0.52%	3.27	2 min. 30 sec.	3 to 5 ins.
Manhole 4 to Manhole 5..	6 "	484 "	0.40%	1.96	4 " 7 "	3⅜ ins.
" 5 to " 6*.	8 "	500 "	0.20%	1.33	.....	

* A slight current was flowing through this reach at the time of discharge.

Flush Tank "M."

Capacity, 230 galls. Time to discharge, 32 seconds.

Reach.	Size of sewer.	Distance.	Grade.	Average velocity, front of discharge, Feet per second.	Duration of maximum discharge.	Maximum depth on invert (approximate).
Tank to Manhole 7.....	6 ins.	575 ft.	2.8%	6.25	.....	Full.
Manhole 7 to Manhole 8.	6 "	260 "	0.5%	3.56	.....	2½ ins.
" 8 to " 9.	6 "	321 "	0.5%	1.86	1 min. 20 sec.	3 "
" 9 to " 10*	8 "	775 "	0.2%	1.17	.....	.....

* This reach was somewhat soiled when measurements were made.

Flush Tank "N."

Capacity, 379 galls. Time to discharge, 1 minute.

Reach.	Size of sewer.	Distance.	Grade.	Average velocity, front of discharge. Feet per second.	Duration of maximum discharge.	Maximum depth on invert (approximate).
Tank to Manhole 11.....	6 ins.	435 ft.	0.4%	3.95	.....	2⅝ ins.
Manhole 11 to Manhole 12	8 "	500 "	0.2%	1.35	2 min. 30 sec.	2¼ "
" 12 to " 13	8 "	500 "	0.2%	0.91	.....	1⅝ "
" 13 to " 14	8 "	500 "	0.2%	0.74	6 min. 3 sec.	1⅜ "

Mr. Folwell. A. Prescott Folwell, Assoc. M. Am. Soc. C. E.—The writer believes that the author takes too literally the wording of the quotation forming his text. He does not think it has ever been generally claimed that "flush tanks" make the separate system practicable, but it has undoubtedly been claimed that flushing does so, and flush tanks are usually recommended and employed, and are cheaper and more reliable than flushing by means of a hose or other appliance when intrusted to careless employees. The writer knows many engineers who do not consider tanks indispensable. In most of the systems which he has designed he has recommended the use of flush tanks, but has suggested other methods for attaining the same end should there be any reason for omitting them. The experience of several of these systems which have for a time dispensed with them will be referred to later.

After a careful reading the writer is in doubt whether the author wishes to challenge the correctness of the quoted statement should flushing be substituted for flush tanks. He claims that "flush tanks * * * are not an essential feature of * * * all separate sewer systems." The writer hopes he will state whether, as the result of his experience, he considers flushing necessary for all separate sewer systems, and also that he will give the minimum grades in the Mount Vernon sewers, the average consumption of water per capita and the probability of ground water entering the pipes to any extent through the joints. The data must be definite on these points, for if it shall prove practicable to omit flushing in any or all systems or parts of systems, many towns contemplating constructing sewers may be spared a considerable expense, both in construction and in maintenance.

In the writer's experience he has omitted flush tanks in the designs in two cases where the grades were considerable, 8% to 12%, and a public water supply already introduced into these streets, and stated his belief that no flushing would be required, but recommended a flushing by hose at intervals of a few months. Both of these steep lines, however, being on the outskirts of cities, are as yet unbuilt and cannot be quoted as examples of either right or wrong construction. If they shall prove to be the latter, flush tanks can easily be added at any time.

In two more cases where conditions were different, experience did not favor the omission of flush tanks. In both these cases the grades were, with few exceptions, those theoretically giving a velocity of 2½ ft. per second, the pipes were carefully graded and jointed and no ground water or mud could enter the sewer through the joints. Flush tanks were provided for at the head of each branch, but not set until the entire plant was completed. Meantime, under pressure from many householders, the authorities permitted connections to be made with the sewers and used as soon as an outlet was provided. In each case these connections were used for two or three months before the tanks

were in operation. At the end of that period an inspection showed that there were many stoppages from house waste, aside from those occasioned by garbage, ashes and old clothes, and that almost every dead end emitted many foul odors. It is possible that this condition would not have existed were all the properties along each sewer branch using it freely. But this is a condition which is not found for several years after construction in many of the lateral branches of a system. Where the grades were steep, in two short sections they exceeded 6%, no stoppages were discovered and there was no odor whatever. Mr. Folwell.

This experience of so short duration cannot be considered as outweighing that of Mount Vernon. But the writer cannot see how a longer existence of the conditions would have improved matters or rendered the conclusions less favorable to the necessity of flushing. The separate system as generally constructed and operated is distinctly a water carriage system and relies for its success upon a sufficiency of water properly applied. If the ordinary house water answers this purpose a great saving may be effected.

William B. Landreth, M. Am. Soc. C. E.—A majority of the separate sewerage systems as designed and built during the past 10 or 12 years have been provided with some means of regular flushing, generally by some form of automatic flushing tank. As is well known, the water passing through a sewer must have at least a certain velocity of flow to carry the heavier particles of sewage and prevent their stranding in the pipe. The minimum cleansing velocity depends upon the size and rate of fall of the sewer, and the quantity of water necessary to secure that velocity must be supplied in some manner if the sewer is to be kept clean. If sufficient water were furnished from house connections or entered the sewer through the joints, artificial flushing would be unnecessary, and the quantity of water supplied from those sources are important factors in deciding if automatic tanks are needed. Both of the sources mentioned are uncertain and their combined flow difficult of determination, owing to the fact that the leakage into the pipes generally decreases with the age of the sewer, while the number of house connections which will probably be made during the early stages of its use can rarely be foretold. There seems to be greater need of frequent and regular flushing during the first year or two of the use of the sewers than later, when the quantity of sewage from the house connections has increased and is sufficient to flush them properly. Mr. Landreth.

In a properly designed system where the sizes and grades of the sewers are such as to give a cleansing velocity to the sewage when the pipes are running half full, as soon as sufficient sewage is flowing to keep the sewer clean, the flush tanks are useful mainly in flushing the upper portions of the system, from the tank to a point in the sewer where a cleansing flow is found. In a system already constructed, the

Mr. Landreth. interval between successive flushings can be readily determined by experiment, and will generally vary in different portions. Where regular flushing of a system of sewers has been found necessary, it is the belief of the writer that the cheapest method of applying the water is by means of automatic flush tanks. With flush tanks holding 300 galls. each, and with an average of four tanks per mile of sewer, the daily quantity of water necessary for one flushing would be 1 200 galls. per mile, or 24 000 galls. for a system of 20 miles, costing probably less than $2 50.

Some separate systems have been designed and built with such low grades that a cleansing velocity could never be obtained, and the flushing of such systems in any manner would be of little use. It has been the practice of the writer to use some form of automatic flush tank on all separate sewers, where water for feeding them could be secured; but he is not prepared to say that they could not have been omitted in some cases without detriment to the operation of the system. Experience gained in the construction and care of several hundred flush tanks, of several forms and under varying conditions, has taught him that some will work and some will not; some will work with a small trickle, while others need to be flooded with a strong stream; some will work for a while, but refuse to operate after a time.

Mr. Le Conte. L. J. Le Conte, M. Am. Soc. C. E.—Most sanitary engineers will agree with the author that automatic flush tanks are not inseparable adjuncts to the separate system. The effects of a 150-gall. flush are all expended a few hundred feet away from the tank, the distance varying with the grade of the sewer. Where grades are light and the flush is most needed they do the poorest work, and some auxiliary means have to be adopted to furnish a long, continued flush, preferably a pipe connection with the city water main. Two cities in California, Oakland and Alameda, the former with 60 000, and the latter with 10 000, population, have strongly contrasting separate systems. Oakland has some 100 miles of pipe sewers on the separate system, without flush tanks. The only storm water allowed to enter the sewers is roof water at and near the upper ends of the streets. The sewers are flushed regularly two to three times a year by means of a line of hose attached to fire hydrants. This is done during the dry season. In the city of Alameda the flush tank system is in use. The tanks are located not only at the upper ends, but also at various intervals along each street sewer down to the outfall into the main intercepter. Sometimes these tanks flush regularly three times a day, and sometimes they do not work at all, but the results seem to remain the same. The writer does not believe in the efficiency of these small flushes; they are impotent and waste a large amount of water to no practical purpose. The tanks are expensive and require constant attention. One good, continuous flush from a water main he considers worth a myriad of tiny flushes from tanks. It is cheaper, more effective and requires less water.

Mr. Le Conte.

The Oakland system compares most favorably with that in Alameda in every respect. The only trouble with choking has been due to roots of trees finding their way through the cemented joints, and finally filling the whole bore of the pipe, often bursting the joints. Many beautiful rows of shade trees had to be cut down on this account. A growth of trees near an aqueduct, sewer or masonry reservoir is a constant menace to the work. The roots of eucalyptus trees near a service reservoir at the Mare Island Navy Yard penetrated through the brick masonry wall, 4 ft. thick, and finally cracked it so badly that a serious leak was developed. On emptying the reservoir, the bottom was found to be covered with a mat of roots, 325 sq. ft. in area and some 12 ins. thick. Where the main root passed through the fissure in the wall, it was like a mammoth saw-blade not over $\frac{1}{8}$ in. thick.

Mr. Odell.

F. S. Odell, M. Am. Soc. C. E.—While all who engaged in the discussion of this paper have expressed the opinion that flush tanks were not essential to the proper working of the separate system of sewers, the author believes the fact remains as stated by Mr. Landreth, that the majority of such systems designed and built during the past 10 or 12 years have been provided with some form of automatic flush tank. This would imply that the designers believed either that they were necessary, or that they were efficient and economical. The author has come to believe they are neither, but that a direct connection with the water mains gives a much better flushing, and in the course of the year uses far less water. His experience has been that flush tanks discharging 200 galls. or less of water fail to flush a sewer properly, especially on the flat grades where flushing is most needed. The grades at Mount Vernon are mostly good, from 0.5 to 6%, the flattest grades being 0.25%, and on all these sewers a good flushing twice a year answers every purpose.

Regarding the cost, it may be interesting to note a comparison with a neighboring municipality, New Rochelle, where flush tanks are used on all dead ends. It has 40 tanks and during the year ending January 9th, 1895, used 5 400 000 galls. of water. Under contract with the water company, it pays at the rate of $12 per annum for each flush tank, using a maximum quantity of 60 000 galls.; for all water used in excess of this quantity payment is made at the rate of 5 cents per 1 000 galls; under this arrangement the water bill for the year amounts to $630 21. Mount Vernon, with double the number of dead ends, must necessarily have paid twice that amount, or $1 260 42. As a matter of fact, the quantity of water used in flushing the sewers of Mount Vernon for the year 1894 was about 500 000 galls., as determined by the quantity pumped during the flushing period, in excess of the normal consumption. This, at the maximum price paid at New Rochelle, would amount to $100, showing a saving of $1 160 on water; while the cost of labor is much less than the cost of inspection of the flush tanks.

Mr. Odell. It is impossible to estimate the per capita consumption of water in a suburban city such as Mount Vernon, where, perhaps, a majority of water-takers use it liberally in lawn sprinkling during the summer season, when the smallest quantity of ground water finds its way into the sewers. Nor can the quantity of ground water reaching the sewers be definitely determined. In the lateral sewers where flushing is most needed, the quantity is very small, as such sewers are located on the high and dry grounds of the city.

The author does not think it judicious to omit provision for flushing in the design or construction of a separate system of sewers. The cost of connecting the water main by a 4-in. branch and gate with the manhole at the dead end of a sewer is about $40, which is a very small item. It is the simplest and best method of flushing the author has yet tried, and he strongly recommends its use. Flushing through a hose from fire hydrants is slow, and consequently much more expensive than by means of a gate on the water main.

Doubtless the automatic flush tank was expected to so cleanse the sewer on which it was placed that no material susceptible of decomposition could remain stranded during the passage of the water from the tank, but observation and experiment seem to discredit such an assumption; consequently, the sanitary condition of a sewer that is effectually flushed two or three times yearly is found to be as good as, or better than, one superficially flushed by small daily discharges from automatic tanks.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

760.

Vol. XXXIV.—September, 1895.

EXPERIMENTS ON THE DISCHARGE OF A 30-IN. STOP VALVE.

By J. WALDO SMITH, Assoc. M. Am. Soc. C. E.

READ AT THE ANNUAL CONVENTION, JUNE, 1895.

WITH DISCUSSION.

In connection with the measurement of the flow of the Pequannock River and the operation of the water supply system built by the East Jersey Water Company for the city of Newark, N. J., it became desirable to ascertain the discharge of certain 30-in. stop valves at the outlets of the storage reservoirs. The author was directed by the chief engineer, Clemens Herschel, M. Am. Soc. C. E., to make the necessary gaugings, a brief description of which, with a tabulation of results obtained, may be of interest to members of this Society. Opportunities for making careful experiments of this kind are comparatively rare, and when they do arise it seems to the author that the results should be placed where they may be of general use to members of the profession, such description being presented in as

brief a form as is consistent with entire clearness and still be sufficiently detailed to serve as a true guide for their further application.

The valve selected for experiment was one of four of the same pattern and workmanship, located in the gate house at the outlet of Clinton Reservoir. Figs. 1 and 2 show the general arrangement of the pipes and gates, the channels of approach and discharge, and the location of the measuring weir. The experiments numbered 1 to 5, inclusive, were made with the gate numbered 1, and the remainder with No. 3.

It will be noticed that in constructing the discharge pipes and gates use has been made of the Venturi principle, by setting the gate at the common apex of the frustums of two cones, enlarging in both directions to 42 ins. in diameter, and terminating in a short piece of straight pipe at the lower end and in a piece of straight pipe and a "flap gate" at the upper end. The flap gate is intended for use should the 30-in. valve become seriously disarranged, necessitating its removal, and was wide open, as shown, at the time of the experiments. The valve was made by the Ludlow Valve Manufacturing Company, of Troy, N. Y., in 1891, and has two parallel cast-iron disks, with faces about 6½ ins. apart. Fig. 3 shows the details of the gate.

The reservoir having been emptied a few days after the experiments were finished, opportunity was thus afforded to make the necessary measurements of the valve. The brass gate rings were found to be 32.96 ins. in diameter, and the brass seat rings 30.22 ins. in diameter. Careful measurements were also made of the cast-iron lug a at the bottom of the disks. The area of the lune-shaped orifice of discharge for different positions of the disks was computed from the above measurements, and the results are given in Table No. 1. There, and in all cases in this paper, "height of gate" refers to the clear height of the bottom of the gate ring above the top of the seat ring, and is represented by the distance b in Fig. 3. The headings of the remaining lines are self-explanatory. Lines 1 and 2 of this table were plotted as ordinates and abscissas, and a diagram constructed, from which the area of the lune-shaped orifice for any gate opening could be read at a glance. Correction for the cast-iron lug a was made by deducting the area of its longitudinal cross-section from the area of the lune-shaped orifice for all heights of gate greater, and the proportional part of such cross-section opposed to the discharge for heights

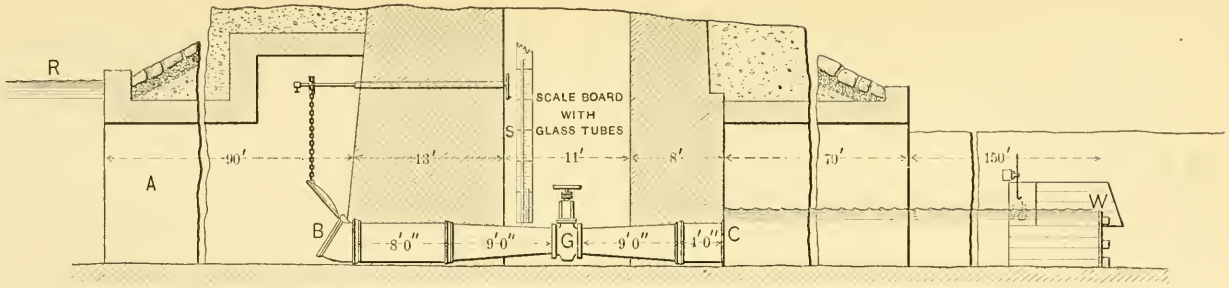


Fig. 1. SECTION THROUGH GATE-HOUSE ARCH-WAY AND WEIR.

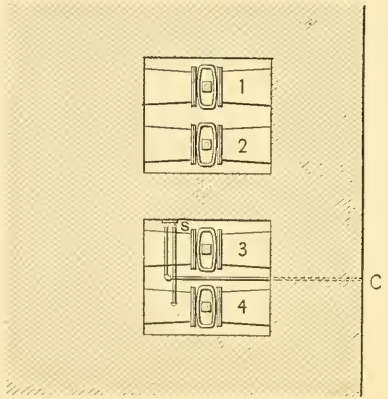


Fig. 2. PLAN OF GATE CHAMBER.

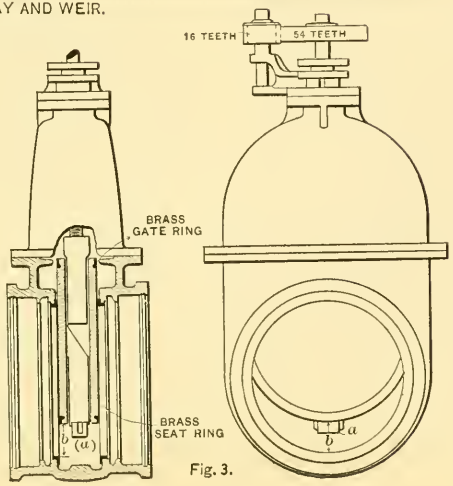


Fig. 3.

of gate less, than the depth of the lug. The areas given in Table No. 2, column 4, are the net areas with the lug deducted.

The height of the gate was observed by means of an indicator, which was carefully rated when the reservoir was empty. To do this the gate was raised to any height and the corresponding distance *b*, Fig. 3, of the bottom of the disk ring above the seat ring was measured and compared with the height shown by the indicator. In all the experiments the gate was set on the opening movement so as to avoid errors from back-lash.

TABLE No. 1.

GIVING AREAS OF LUNE-SHAPED ORIFICE IN A 30-IN. STOP VALVE FOR DIFFERENT HEIGHTS OF GATE.

Height of gate. Inches.....	1.	3.	5.	8.	12.	16.	20.	23.	25.	27.5	30.2
Area of opening. Square feet.	0.101	0.451	0.869	1.496	2.308	3.070	3.761	4.213	4.474	4.738	4.901
Proportional height of gate...	0.033	0.099	0.165	0.265	0.397	0.530	0.662	0.761	0.828	0.911	1.000
Proportional area of opening..	0.021	0.092	0.177	0.305	0.471	0.626	0.767	0.860	0.913	0.967	1.000

In conducting the experiments the water was drawn from the reservoir *R* (see Fig. 1), through the archway *A*, discharged through the gate *G*, and measured over the weir *W*. The height of the water was observed at *B*, just at the inlet of the pipes, by tapping one of the unused pipes, No. 4, above the gate, and piping to a glass tube on the scale board *S*. A pipe through the down-stream wall of the gate-house served to bring the water from *C*, just at the outlet of the pipes, into the gate chamber, where its height was observed on the same scale. The pipes were so arranged that a full stream of water could be run through them, to ensure freedom from entrained air before any observations were made. After passing the pipe and gate the water flowed through an archway and open rock cut about 220 ft. to the weir *W*.

The crest of the weir was of hard pine, the plane of the up-stream side being vertical, and the edge presented to the current being sharp; the down-stream side was chamfered so that the water in no case touched any part of the weir after passing the up-stream edge. The sides of the channel of approach were of planed matched boards, those at the ends of, and above, the weir, extended a short distance down stream from the crest, thus preventing the expansion of the sheet after passing the weir. Provision was made for a free circulation of air under

the sheet. The depth on the weir was measured by means of a hook gauge securely fastened to a large timber, whose ends rested in the rock sides of the cut. The hook gauge pail, placed on a movable bracket, was connected by means of a rubber tube and a brass pipe with an orifice in a brass plate set in the side of the flume. This orifice was normal to the plane of the plate, which was perfectly smooth and true, and was about 0.20 ft. below, and 6 ft. up stream from the line of the crest of the weir. The end contractions were suppressed, and the length of the weir was exactly 12 ft. The mean depth of the channel of approach below the crest of the weir was 4.07 ft. No racks were used to quiet the water, but the sheet went over the weir in good form, although with the greatest depths, 2 ft. and above, there was an oscillation of the surface level, amounting to 0.13 ft., which was not, however, communicated to the surface of the water in the hook gauge pail.

TABLE No. 2.

Record of Observations and Results of Computations.

1	2	3	4	5	6	7	8	9	10	11
Date, 1892.	No.	Height of gate. Inches.	Area of opening A. Square feet.	Depth on Weir. Feet.	Water passing weir Q. Cubic feet per second.	Reading of Head Gauges. Upper. Feet.	Reading of Head Gauges. Lower. Feet.	Head on gate H. Feet.	Coefficient of discharge, $\frac{Q}{A\sqrt{2gH}}$	Quantity in column 6 reduced to head of 9 ft.
Aug. 23...	1	11.9	2.29	1.142	49.30	64.60	53.79	10.81	0.816	44.98
" ..	2	8.9	1.68	0.934	36.35	64.58	53.63	10.95	0.815	32.95
" ..	3	5.9	1.07	0.713	24.10	64.58	53.45	11.13	0.842	21.67
" ..	4	3.0	0.45	0.443	11.75	64.58	53.21	11.37	0.965	10.45
" ..	5	0.9	0.09	0.228	4.35	64.58	53.00	11.58	1.771	3.83
" ..	6	3.3	0'51	0.452	12.10	64.58	53.20	11.38	0.877	10.76
" ..	7	7.2	1.33	0.808	29.15	64 58	53.50	11.08	0.821	26.27
" ..	8	11.2	2.15	1.075	45.02	64.57	53.64	10.93	0.790	40.85
Aug. 26...	9	1.3	0.15	0.225	4.25	62.78	58.00	9.78	1.130	4.08
" ..	10	3.6	0.57	0.458	12.40	62.79	53.22	9.51	0.877	12.03
" ..	11	6.6	1.21	0.712	24.08	62.79	53.45	9.34	0.811	23.61
" ..	12	9.6	1.82	0.916	35.25	62.79	53.59	9.20	0.796	34.86
" ..	13	12.6	2.44	1.106	47.00	62.79	53.69	9.10	0.796	46.74
" ..	14	15.5	2.97	1.292	59.50	62.78	53.84	8.94	0.835	59.70
" ..	15	18.5	3.51	1.529	76.95	62.77	54.02	8.75	0.924	78.04
" ..	16	21.5	3.99	1.781	97.05	62.76	54.18	8.58	1.035	99.40
" ..	17	24.5	4.40	2.081	123.25	62.67	54.34	8.33	1.210	128.11
" ..	18	30.2	4.90	2.320	145.95	62.66	54.46	8.20	1.297	152.90
Aug. 30...	19	30.2	4.90	2.100	125.00	60.54	54.36	6.18	1.279	150.85
Sept. 4..	20	25.5	4.52	1.494	74.25	56.73	54.06	2.67	1.254	136.32
" ..	21	22.8	4.18	1.332	62.30	56.73	53.96	2.77	1.117	112.30
" ..	22	27.5	4.74	1.556	78.95	56.72	54.11	2.61	1.286	146.61
" ..	23	80.2	4.90	1.584	81.15	56.72	54.14	2.58	1.286	151.57

The flow over the weir was computed by the Francis formula, $Q = 3.33\ L\ H^{\frac{3}{2}}$, in which Q is the discharge in cubic feet per second, L is the length of the weir in feet, and H is the depth on the weir in feet. Correction was made for velocity of approach.

The hook gauge was observed every minute, and the head gauges, which scarcely varied at all during a single experiment, every five minutes.

Table No. 2 gives a tabulation of the observations and results. Col-

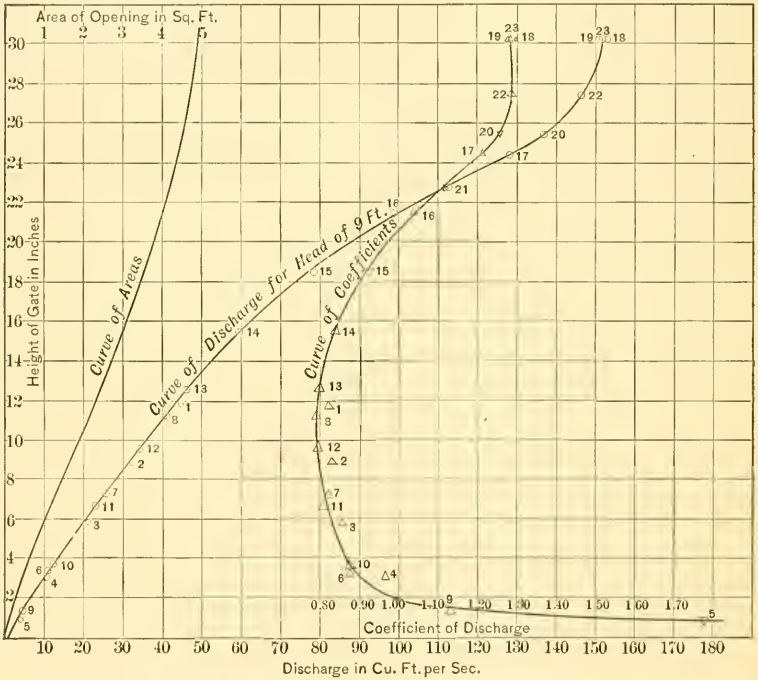


FIG. 4.

umn 3 gives the clear height of the bottom of the disk ring above the seat ring, and is represented by the distance b in Fig. 3. Column 4 gives the net area of the lune-shaped opening, correction having been made for the lug at the bottom of the disks. Column 9 shows the difference between the height of water at the inlet and outlet of the 42-in. pipe, points B and C in Fig. 1. The headings of the remaining columns are self-explanatory. The results of the experiments are more clearly shown by the curves in Fig. 4. A good degree of regularity will be

noted in the plotted points for experiments made with different gate openings, and with a considerable variation in head, thus proving the accuracy of the experiments.

In experiments Nos. 1 to 5, some doubt existed as to the indicator being in good working order, and in No. 18 the depth on the weir was too great for a good measurement, the water striking the cap timbers; consequently those experiments are not entitled to as much weight as the others. The coefficients given in column 10 are not, strictly

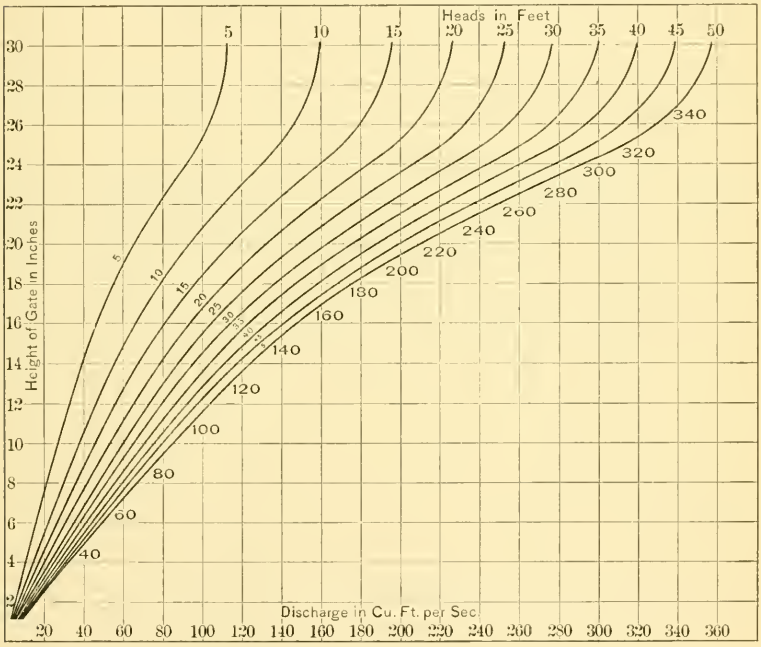


FIG. 5.

speaking, coefficients of discharge of the gate alone, but of the gate and the short lengths of connecting pipes. The head might have been observed immediately above and below the gate, but it would have been difficult to get satisfactory readings, and, in the present case, would have served no practical purpose.

The upper half of the curve of coefficients is quite different from that found by E. Kuichling, M. Am. Soc. C. E., for a 24-in. stop valve.* This disagreement is to be expected when the differing conditions are

* See the *Transactions* of the American Society of Civil Engineers, Vol. XXVI, p. 439.

considered. In Mr. Kuichling's case, as the disks were raised, the limit of the coefficient with the full opening of gate was infinity, while in the present case the coefficient approached a certain definite limit. Up to half gate there is a similarity in the curves, and as near an agreement as could be expected considering that the gates are used in different ways, and for a different purpose.

In applying these results to approximately the same sizes of gate set in the same manner as the one experimented upon, Table No. 3, which is computed from the curves, will be found useful.

TABLE No. 3.

PROPORTIONAL VALUES FOR HEIGHT OF GATE, AREA OF LUNE-SHAPED OPENING, AND DISCHARGE.

Proportional height of gate.....	0.125	0.250	0.375	0.500	0.625	0.750	0.875	1.000
" area of opening....	0.125	0.287	0.443	0.593	0.729	0.851	0.946	1.000
" discharge.....	0.086	0.178	0.274	0.384	0.533	0.734	0.936	1.000

In Fig. 5 is given a diagram of the discharge of the 30-in. gate for each 5 ft. of head up to 50 ft., based on these experiments.

DISCUSSION.

E. KUICHLING, M. Am. Soc. C. E.—The data contained in the paper deserve careful attention as they relate to a very large valve, while the similar data given in the familiar manuals on hydraulics were derived from experiments with very small valves. In character the experiments now submitted are like those made a few years ago with a 24-in. stop-valve at Rochester, N. Y.,* but as the author states that the values of certain coefficients found by him are considerably different from those obtained in the paper referred to, the speaker desired to make some comments concerning them. Mr. Kuichling.

The coefficients mentioned by the author as not being in accord with those found from the aforesaid experiments with a 24-in. stop-valve are simply the coefficients of discharge computed from the given data on the assumption that the orifice presented by the valve opening is in a comparatively thin wall and is fully submerged. These coefficients were originally computed for the purpose of a collateral analysis respecting the flow of water in pipes, and were submitted in the paper mentioned in the hope that they might be utilized by others who had more time than the speaker to complete such an analysis. To present the matter clearly, the following notation was adopted: $Q =$ discharge through valve in cubic feet per second; $A =$ area of orifice in square feet; $H =$ total observed loss of head in feet in the entire apparatus, as given by the author in column 9 of Table No. 2 of the paper, this being the difference in level between the water surface in the reservoir and that in the flume above the measuring weir; $2g =$ twice the acceleration due to gravity $= 64.4$ ft. per second; $y =$ loss of head due to the passage of the volume Q through the valve orifice alone, while $x =$ loss of head in the remainder of the apparatus, so that $x + y = H$; and, finally, $\mu =$ coefficient of discharge for the valve opening considered as a submerged orifice in a comparatively thin wall. There then exists the well-known relation:

$$(1) \ldots\ldots\ldots\ldots Q = \mu A \sqrt{2 g y}, \text{ whence } \mu = \frac{Q}{A \sqrt{2 g y}}.$$

It should be remarked that it was on this basis that the coefficients μ were computed in the speaker's paper already mentioned. The author, on the other hand, has computed coefficients on the basis of taking the entire observed loss of head H in the entire apparatus, instead of only the portion y relating to the valve alone, and has made: $\mu = \frac{Q}{A \sqrt{2 g H}}$. Obviously the resulting values of μ cannot agree with those derived by considering the partial loss of head y

*See the *Transactions* of the American Society of Civil Engineers, Vol. xxvi, p. 439.

Mr. Kuichling. alone; and it now becomes of interest to ascertain the proper values of y in the author's experiments, whereupon a new computation of the coefficients μ will enable a proper comparison to be made with those for the 24-in. valve.

To determine the proper values of y in the case under consideration, it is advisable to deduce from the given data the relation between Q and H when the 30-in. valve is fully open, in which event $y = 0$ and $H = x$. For this purpose experiments Nos. 18, 9 and 23 of Table No. 2 are at once available, and as the author's values of μ are practically constant, the following equation is obtained therefrom directly: $Q = c\sqrt{H}$, whence:

$$(2)\ldots\ldots\ldots H = x = \left(\frac{Q}{c}\right)^2 = 0.00039106\ Q^2.$$

This expression accordingly represents the relation between the loss of head in the apparatus exclusive of the valve (which is fully opened) and the discharge Q, and by constructing the parabolic curve represented thereby with x as ordinates, and Q as abscissas, the value of x for any value of Q given in the author's Table No. 2 may then be quickly found. The difference between these values of x and the corresponding values of H will then give the proper values of y sought, whereupon their substitution in equation 1 will yield the values of the coefficient μ which may properly be compared with the ones obtained from the 24-in. valve. By this process the values given in Table A were obtained.

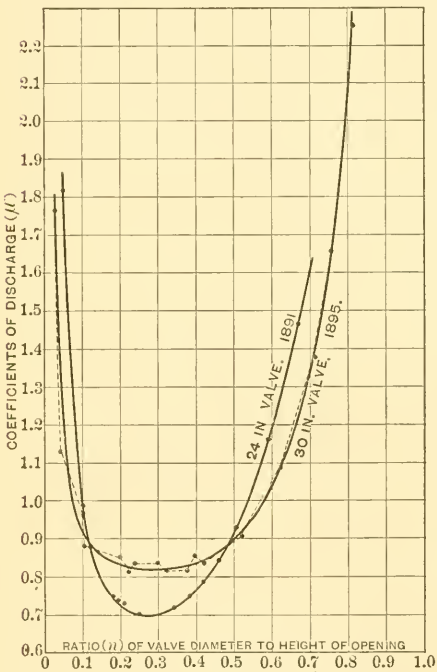


FIG. 6.

If the values of μ in column nine of the foregoing table are plotted as ordinates to the corresponding values of n in the second column as abscissas, the line connecting the several points thus obtained will be seen to be somewhat irregular; but by passing a curve through the line so as to cut off the irregularities, this curve will be found very similar in shape to that which was found for the case of the 24-in. valve. For the purpose of exhibiting this similarity, the two curves are shown in Fig. 6.

Mr. Kuichling.

It may be remarked that the irregularities indicate some slight errors in the original observations which are closely revealed by the above development of the coefficient μ; also that the results obtained from the observations on the 24-in. valve by means of the mercurial pressure-difference gauge, described in the paper mentioned, are remarkably free from irregularities. The difference between the numerical values of μ for the 30-in. and 24-in. valves is clearly exhibited

TABLE A.

Showing Observed Data and Results of Original and Revised Computations.

1	2	3	4	5	6	7	8	9
Number of experiment.	Ratio of diameter of valve to height of opening m.	Area of opening A in square feet.	Discharge through opening Q in cubic feet per second.	Total observed head H in entire apparatus. Feet.	Computed loss of head x in apparatus except valve. Feet.	Computed loss of head y in valve alone. Feet.	Coefficient of discharge μ computed with H.	Coefficient of discharge μ computed with y.
1.....	0.394	2.29	49.30	10.81	0.94	9.87	0.816	0.854
2.....	0.295	1.68	36.35	10.95	0.51	10.44	0.815	0.835
3.....	0.195	1.07	24.10	11.13	0.21	10.92	0.842	0.849
4.....	0.099	0.45	11.75	11.37	0.05	11.32	0.965	0.967
5.....	0.029	0.09	4.35	11.58	0.01	11.57	1.771	1.771
6.....	0.109	0.51	12.10	11.38	0.06	11.32	0.877	0.879
7.....	0.238	1.33	29.15	11.08	0.32	10.76	0.821	0.833
8.....	0.371	2.15	45.02	10.93	0.79	10.14	0.790	0.819
9.....	0.043	0.15	4.25	9.78	0.01	9.77	1.130	1.130
10.....	0.119	0.57	12.40	9.51	0.06	9.45	0.877	0.882
11.....	0.218	1.21	24.08	9.34	0.21	9.13	0.811	0.821
12.....	0.318	1.82	35.25	9.20	0.49	8.71	0.796	0.818
13.....	0.417	2.44	47.00	9.10	0.86	8.24	0.796	0.836
14.....	0.513	2.97	59.50	8.94	1.38	7.56	0.835	0.908
15.....	0.612	3.51	76.95	8.75	2.30	6.45	0.924	1.076
16.....	0.712	3.99	97.05	8.58	3.69	4.89	1.035	1.371
17.....	0.811	4.40	123.25	8.33	5.93	2.40	1.210	2.253
18.....	1.000	4.90	145.95	8.20	8.31	0.00	1.297	∞
19.....	1.000	4.90	125.00	6.18	6.10	0.00	1.279	∞
20.....	0.844	4.52	74.25	2.67	2.15	0.52	1.254	2.530
21.....	0.755	4.18	62.30	2.77	1.51	1.26	1.117	1.655
22.....	0.911	4.74	78.95	2.61	2.42	0.19	1.286	4.762
23.....	1.000	4.90	81.15	2.58	2.58	0.00	1.286	∞

in the diagram; and the only comment which can now be made on the results is that the failure in agreement points out the fact that the laws which govern the motion of water in pipes are not yet definitely known. Owing to this circumstance, the paper cannot fail to be a welcome and valuable addition to the knowledge of the subject, and the speaker hoped that further contributions of a similar character will be offered.

CORRESPONDENCE.

Mr. Smith. J. WALDO SMITH, Assoc. M. Am. Soc. C. E.—Mr. Kuichling has treated the subject so exhaustively that there is little more to be said. The author did not compute the coefficient, μ, for the reason that in the work which called forth the experiments it would have served no practical purpose, for the result wanted was the discharge of the gate and connecting pipes with any elevation of water in the reservoir, and he was satisfied to present the facts just as they came to him, with a full explanation of the method of making all the observations, so that any one could adapt the results to suit the case in hand. He is very glad, however, that Mr. Kuichling has taken so much trouble to reduce the experiments to a form to make them directly comparable with his own and others, for their whole range.

In regard to the greater regularity of the coefficient, μ, as given by the experiments of Mr. Kuichling, the author thinks this regularity is not so much due to the use of the pressure-difference gauge as to the fact that the μ, as he understands from a careful perusal of the above experiments, is the result of a computation from adjusted quantities, and not from the actual experimental observations, *i. e.*, in the formula* $\mu = \frac{Q}{F\sqrt{2gy}}$, the loss of head appears to have been adjusted by plotting the experimental values as obtained from the pressure-difference gauge, drawing a curve, and reading new values of y, from this curve, thus eliminating any irregularities or errors in the original observations; Q is computed by formula, so the resulting quantity, μ, would necessarily show great regularity.

* See the *Transactions* of the American Society of Civil Engineers, Vol., xxvi., p. 449.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Note.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

761.

Vol. XXXIV.—September, 1895.

CONSTRUCTION OF SUBSTRUCTURE FOR LONESOME VALLEY VIADUCT, KNOXVILLE, CUMBERLAND GAP AND LOUISVILLE RAILROAD.

By Gustave R. Tuska, Jun. Am. Soc. C. E.

Read at the Annual Convention, June, 1895.

WITH DISCUSSION.

About 60 miles from Knoxville, Tenn., the Knoxville, Cumberland Gap and Louisville Railroad crosses Lonesome Valley on a viaduct about 800 ft. long and at a maximum height above the valley of 135 ft. Formerly the railroad crossed this valley on a wooden trestle which collapsed while a freight train was upon it, its collapse being due to its meager bracing. The railroad company having decided to reconstruct this viaduct, William Barclay Parsons, M. Am. Soc. C. E., the consulting engineer of the road, made designs and specifications for the work. The structure is a single-track steel viaduct of fourteen 30-ft. and five 70-ft. deck spans, the 70-ft. spans being latticed girders, while plate girders are used for the 30-ft. spans. The towers are of the ordinary type, in which all connections are riveted. Besides the two abutments, there were required to be constructed 36 piers for the towers of the viaduct.

The only rock to be found in this locality is a light limestone, which deteriorates rapidly. Some bridge abutments on this railroad built of this stone but a few years ago show the stone in a very poor condition. The bids for masonry, built of this material, ran from $12 to $16 per cubic yard. A much better substructure could be constructed by using concrete made from this same limestone. Furthermore, concrete made with the best sand and Portland cement could be had for $8 per cubic yard paid in receiver's certificates, which at that time was equivalent to about $7 cash, a saving, therefore, of about 50% in the cost of the concrete over that of limestone masonry. The reasons for this reduced cost appear at once. With the concrete construction no derricks and hoisting plant were required, as would have been the case with masonry. The cost of this plant would have been high, as the work was spread out over such a large area and materials could be delivered only at the abutments. No derricks were even required in this case to set bridge-seats and cap-stones; for in the place of the former, ¾-in. steel plates were used, and the piers were finished on top by a steel plate 4 ft. square and ½ in. thick instead of using coping stones. Furthermore, for the concrete construction the commonest labor could be employed, in this case colored labor being used for which the contractors paid $1 per day of 11 hours' work. Since, therefore, a better substructure at a less cost could be obtained by using concrete instead of stone masonry, it was decided to build the abutments and piers required for this viaduct of Portland cement concrete. The author was thereupon employed by the railroad company as resident engineer to superintend the construction of the substructure.

The abutments were both built of the U design. The south abutment has a maximum height of 26 ft. from foundation to top. It is 25 ft. long at the bottom of the face, while the wing walls are each 28 ft. 6 ins. long at the top. The north abutment has a maximum height of 27 ft. from foundation to top, is 26 ft. long at the bottom of the face, and the length of the wing walls is 26 ft. 6 ins. at the top. The faces and sides of the abutments have been given a batter of 1 in. to the foot, and both abutments finish to a thickness of 5 ft. at the bridge seat, while the wing walls finish on top to a width of 3 ft. 6. ins. The piers were designed 4 ft. square on top, and the four sides have a batter of 1 in. to the foot. These piers vary in height from 5 to 16 ft.

On digging for a foundation for the various pedestals and abut-

ments, there was found in most cases a limestone rock at a depth of from 4 to 12 ft. below the surface. Where the piers rested on a rock foundation, the batter of 1 in. to 1 ft. gave a sufficient area of base. In the case of some of the piers at the bottom of the valley, where no rock was found at a reasonable depth, a good earth foundation was used, and the dimensions of the bottom were increased, so that in no case would the maximum pressure be greater than 1 ton per square foot. Where pockets of clay were encountered in the foundations, as occurred quite often, the material was excavated and the pockets filled with concrete.

After cleaning the bottom of all dirt and loose rock, the molds were built in place. These molds consisted of 2-in. tongued and grooved plank, braced by posts of 2 × 10-in. plank placed about 3 ft. center to center for the abutments, and at each corner for the small piers. At the corners one side was dapped into the other, so as to avoid any great leakage from the molds. The posts were then braced by batter posts from the surrounding earth. In the case of the piers a square frame was then dropped over the molds and securely spiked to the posts, to hold the mold well together. With the abutments the molds were built up as the work of laying the concrete progressed. In the case of the north abutment, the mold was built in 6 to 7-ft. sections, and held together by ¾-in. iron rods which passed through the concrete and which were left in the finished abutment. By this arrangement, much less planking was required, as the lower sections were taken off and used on the upper part of the structure as the work progressed.

The inside of the molds was painted with a thin coat of crude black oil, so that there would be no adhesion of the wood to the concrete. The same mold was used for a number of piers. Since all piers had the same batter, it was only necessary to add a few planks to the bottom of the old mold or to take a few away to bring it to the required height. In the case of the earth foundation before referred to, the pier was built in two parts, the upper portion having the same batter as the other piers, viz., 1 in. to 1 ft., and the lower portion having a sufficient batter to give the required area of foundation. The proportions of materials used in the concrete were 1 part cement, 2 parts sand, and 5 parts broken stone. The cement was the Atlas Portland brand. The best sand to be found in that locality was that

from the bed of the Tennessee River, which was used. The specifications for sand and cement were the same as adopted by the Department of Docks of New York City. Broken stone was obtained cheaply from railroad cuts, which were being worked at both approaches to the viaduct, the rock being crushed and used on the work. The rock was a strong limestone.

About ¾ yd. of concrete was made on each platform at a time, one barrel of cement being taken as the unit of measure; each batch containing 1 barrel cement, 2 barrels sand and 5 barrels broken stone. The method of mixing was as follows: 2 barrels of sand were spread out evenly in the mortar box and 1 barrel of cement was then spread over the sand. The mixing was done in the usual manner by hoes, and when completely mixed, about 12 galls. of water were added. The amount of water varied with the freshness of the cement and the temperature. The stone was measured in wheelbarrows, built with four sides, so that each one would contain one barrel. The stone was dumped and spread along the front of the box to a width of 4 ft. and a depth of 6 ins. After watering the stone, the mortar was thrown on it, and then both were well mixed by turning with shovels. Usually, when the material was twice turned, it was found to be sufficiently mixed to be shoveled into the wheelbarrows and taken to the molds. This shoveling the mixture into the wheelbarrows ensured its being well mixed, as it was thereby turned twice in addition to the previous mixing. The layers of concrete in the molds were from 8 to 10 ins. By passing the blade of a shovel between the mold and the concrete, a small portion of the cement mortar was forced against the mold, thereby giving the finished abutment a smooth appearance with no stones showing. In fact, the effect of this "cutting of the sides" has caused the criticism that the structures have the appearance of too rich a concrete having been used. The tamping of the concrete was done with rammers weighing from 30 to 40 lbs. Too much tamping is to be avoided, as it separates the mortar from the stone, bringing the former to the surface. An excess of water is also detrimental to the concrete, as it brings a layer of cement which has already set to the surface, forming a point of weakness in the finished block, unless it is well scraped off before the next layer is deposited. The upper surfaces of abutments and piers were finished with a ½-in. layer of mortar, consisting of 1 part of cement to 2 parts of sand. Two days after the

completion of the pier, the molds were removed, and the pier had the appearance of being one solid piece of stone. The concrete was protected from the sun's rays by twigs, and watered twice every day for a week after the molds were removed, as this was found to improve it greatly. One abutment contains 321 cu. yds. of concrete. In such large masses, as the work of laying the concrete had to be discontinued a number of times, it was found advisable to build the mass in sections, arranging them so that one section would always overlap the preceding sections considerably.

Before beginning work each day, the surface of the concrete previously laid was roughened with a pick, then cleaned and well moistened, a layer of mortar laid, and the new concrete deposited on this bed. When the molds were taken off it was found that where one day's work joined that of another, lines could be noticed, and in the case of the south abutment these lines were inclined, sloping at about the angle of repose of the wet concrete. At the north abutment temporary partitions were therefore inserted in the mold, and these lines became vertical, each section now being a trapezoidal block. After the concrete was set, some of these joints were cut into, and the concrete there was found to be as solid as in the remainder of the mass.

The non-porosity of the concrete was well shown by the following: On account of the delay in receiving the anchor bolts for the abutment, wooden cores were placed in the concrete in their stead. When the abutment was completed these cores were withdrawn and the holes filled with water and covered with boards and twigs, to protect them from the sun's rays. It was found that, after the concrete had set well, the amount of water in these holes lessened very slowly; in fact, the difference of the levels of the water, from day to day, could not be noticed.

A gang of men would mix and lay about 40 cu. yds. of concrete in a day's work of about 11 hours, when not delayed by lack of materials. The gang was made up as follows:

Filling sand barrels and handling water	1
Filling rock barrows	2
Mixing sand and cement	4
Mixing stone and mortar	4
Wheeling concrete	2
Spreading concrete in the molds	1
Tamping concrete in the molds	1
Foreman	1
Total	16

It was found from actual measurement that 1 cu. yd. of concrete well rammed and set, made up in the proportion by volume of 1 part cement, 2 parts sand and 5 parts broken stone (the broken stone being about 2 ins. in largest dimension), consisted of 1.25 barrels of cement, 10 cu. ft. of sand and 26.5 cu. ft. of broken stone.

The concrete was laid by contract; the price paid being $8 in receiver's certificates, equal at that time to about $7 cash. At this figure there was a small profit in the work in that locality, sand costing 30 cents per cubic yard, and cement $2 80 per barrel, delivered at the work. The total amount of concrete laid in the 36 piers and two abutments was 926 cu. yds.

DISCUSSION.

Mr. Morison. GEORGE S. MORISON, President Am. Soc. C. E.—There are classes of work which engineers prefer to have done by entirely unskilled labor rather than with skilled, because the former will do what is wanted and the latter will not. It is possible that for this reason the cheap negro labor was the best that could have been engaged for the work described in the paper.

Mr. Emery. CHARLES E. EMERY, M. Am. Soc. C. E.—Concrete has been used for reservoirs of considerable size in India, many of which are covered. A covered concrete reservoir was built with much care a number of years ago in Ceylon, but after a time large cracks appeared. An investigation was made, which indicated that they were probably caused by expansion due to temperature changes. A portion of the reservoir had been covered with earth before the cracks appeared, and the trouble was remedied by covering the entire reservoir with earth, so as to have all the concrete equally protected, and by closing the joints with cement covered with a preparation of asphalt.

Mr. Crowell. FOSTER CROWELL, M. Am. Soc. C. E.—Concrete is very largely used in hot climates with satisfactory results. Some years ago the speaker had occasion to examine the concrete retaining walls in use on the island of Jamaica, which had been in service for many years, and were found to be intact and in a fine state of preservation; they were important structures 30 to 40 ft. high in places, supporting a railroad embankment, and had been subject to great heat and to all the destructive influences of weather except frost. It is customary to consider the weak point of concrete to be the damaging effect of frost, but the speaker failed to see why this argument against concrete does not

apply equally to cut stone masonry. The effects on a mass of concrete produced by temperature or moisture are due to imperfections in mixing, and if the mixing and ramming are properly done, the arguments relating to moisture and freezing no longer apply. Mr. Crowell.

F. V. Abbot, M. Am. Soc. C. E.—The author's method of finishing the surface of the concrete so as to avoid cavities, by running a shovel down between the mold and the concrete, had been used by the speaker, but he considered as a better plan that employed by Captain Warren, Corps of Engineers, U. S. Army, at the gunlift battery at Sandy Hook, New York Harbor. The mortar for the concrete was prepared as usual, but a certain proportion of each batch was reserved for facing before the broken stone was mixed in. This mortar was placed along the bottom of the mold in a narrow pile sloping up against the sides as steeply as it would stand. The mold was then filled in with concrete up to about the level of the top of the pile of mortar, and the whole was rammed. The next batch of concrete was then added, but instead of filling it out against the mold the edges of the layer were sloped back from the mold at an angle of about 45°. When this batch had been spread and rammed the mortar reserved from it was placed in the triangular space left between it and the mold, and was carried up above the top of the layer, sloping back toward the sides of the mold again. In this way the mass was made homogeneous, and the surface smooth and true. The method was very rapid and insured the absence of stones near the face of the work so completely that there was no need of subsequent work to make it even and straight. Mr. Abbot.

On another part of the work the molds were taken down just as soon as the cement had set sufficiently, and a surface coat of mortar was then applied with a plasterer's float. This was much less satisfactory, and large pieces of the surface have scaled off. This will continue, because the mortars composing the coating and the concrete are of different compositions and have in consequence different rates of expansion and contraction with changes in temperature. A similar attempt to float or plaster the surface of concrete after it had taken a partial set failed at Fort Sumter, S. C.

Bernard R. Green, M. Am. Soc. C. E.—In building fortifications at Portland, Me., as long ago as 1872, the method of constructing concrete in a mold of planks, properly faced on the inside to prevent adherence of the concrete, was successfully practiced on a large scale. Where a very strong concrete was not necessary, American cement was used for it, but the exposed surface of such concrete was found to withstand freezing weather poorly and so was given a durable facing by plastering the inside of the mold a little in advance of the layers of concrete with a Portland cement mortar, which made an excellent protecting surface. These works have been exposed to the severe weather of that latitude for twenty years, and are still in good Mr. Green.

Mr. Green. condition. The method is entirely practicable and satisfactory if the fresh mortar and fresh concrete are not allowed to set separately, but harden into a monolith. The thickness of the Portland cement mortar coating in the fortifications probably varied from a small fraction of 1 in. to 4 ins., as no effort was made to produce a uniform coat, but simply to keep the stones back from the surface. A sufficient quantity was applied, however, to make an average thickness of about 2 ins. Concrete may be used as a substitute for stone or brick masonry up to the limit of the strength of the cement in any climate, when intelligently made and laid. Good concrete can be made with inferior materials and the cheapest labor under the direction of an engineer specially skilled in such work.

CORRESPONDENCE.

Mr. Parsons. William Barclay Parsons, M. Am. Soc. C. E.—The concrete work described in this paper is interesting from several facts. It is one of the largest pieces of pure concrete masonry used in railroad practice in the United States; the superstructure rests directly upon it without intermediate capstones; there was no cut stone used at all, the only stone used being the broken stone in the work; and it was put up very successfully by the most unskilled form of labor.

The valley was crossed originally by a high wooden trestle, which collapsed under a passing freight train three years ago. After the failure of the trestle, trains were run around the break by constructing a steep gradient on each side of the valley with a trestle of moderate height in the center. A year ago it was decided to replace the trestle with a steel viaduct. There was no stone along the line of the road fit for use in the masonry, the only variety being a limestone, which disintegrated after about three years of exposure.

As consulting engineer of the company, the writer therefore advised the use of concrete in the abutments and piers of the proposed structure. This recommendation met with strong opposition from the chief engineer and the superintendent of the road, who, while admitting the good quality of high-class concrete work, such as exhibited by the Department of Docks in the city of New York, nevertheless had a firm belief that as the contractors and laborers in the South were wholly unacquainted with the manufacture of first-class concrete, it would be impossible to make satisfactory work. The president of the road, however, followed the advice of the consulting engineer, relieving the chief engineer of the responsibility of the foundations and placing it upon the consulting engineer.

Bids were called for the work, and of all the contractors that bid but one was found who had sufficient courage to make a lower price on concrete than on cut stone masonry, and this bid diminished the

cost of the work several thousand dollars, the price that he named for the concrete being the exact amount that the consulting engineer had estimated. Mr. Parsons.

The consulting engineer placed the work in the hands of the author, who took charge of the contractor's men, none of whom had made any concrete, except the cheap lime concrete used in the foundations of small buildings in the South. The men, therefore, were of the most unskilled class possible. In spite of this, however, the author succeeded in constructing both the abutments and piers of concrete, the work comparing favorably with the best work that has been turned out by the Department of Docks in New York. In fact, the success was so great that both the engineer and the superintendent of the road have since admitted the excellence of the construction and are advising the use of concrete for future work, while the contractor who constructed the piers has stated that he will never use cut stone masonry again, if he can help it.

As another instance of the excellence of concrete, the writer desires to mention an experience with some piers for a bridge across the Wabash River, the superstructure of which he is now changing from wood to iron. Some years ago the stone masonry commenced to show signs of disintegration and the piers were incased in beton. The new steel bridge required the tops of the piers to be cut off, due to the change of bridge seat. It was found impossible to split or break the concrete, so that the workmen were obliged to drill holes and blow off the tops of the piers by small charges of gunpowder; in other words, the concrete was in far better condition than any stone masonry could possibly be.

George H. Pegram, M. Am. Soc. C. E.—The value of concrete for bridge piers and foundations is being generally recognized. The writer has used it unhesitatingly for very large bridges with as yet no indication of insufficient strength or want of durability. He has, however, heard of some cases of failure in other work, which could generally be explained on the ground of poor materials or careless workmanship. Mr. Pegram.

He has always used Portland cement, mixing it generally with sand and broken stone in the proportions of one, three and six. Sandstone is apparently better adapted for concrete than limestone. The mortar will hold to it more tenaciously, and it is free from incipient cracks which are apt to be present in limestone. Instead of screening the broken stone, it seems sufficient where sandstone is used to reduce the sand by the amount of detritus contained, thus reducing the cost without sensibly diminishing the strength.

The non-porosity of concrete, which the author's experiment illustrates, demands constant consideration in the execution of the work. Whenever construction is stopped for an interval of time sufficient to allow the mortar to take a partial set before work is continued, surfaces should be left rough and at right angles to the direction of sub-

Mr. Pegram. sequent pressures. On account of this non-porosity, it is very difficult to make plastering stick to these piers unless it is put on before the concrete has set, which is generally impracticable. It seems better to make the form tongued and grooved and surfaced inside, and then take precautions for keeping the stones from getting to the outside of the pier. This can be done by endeavoring to make the outside of the pier of mortar.

In large bridge piers, it is essential to put diagonal bracing from top to bottom to keep the forms in shape; otherwise a twist in the pier or other deformity may develop.

All cement used in concrete should be carefully tested before being used, and the effect of the water with which it is mixed should be carefully determined.

The appearance of the work is greatly benefited by the use of natural stone copings. Where these are used it is generally well to leave the lower bed rough, to give it a better hold on the concrete against lateral displacement.

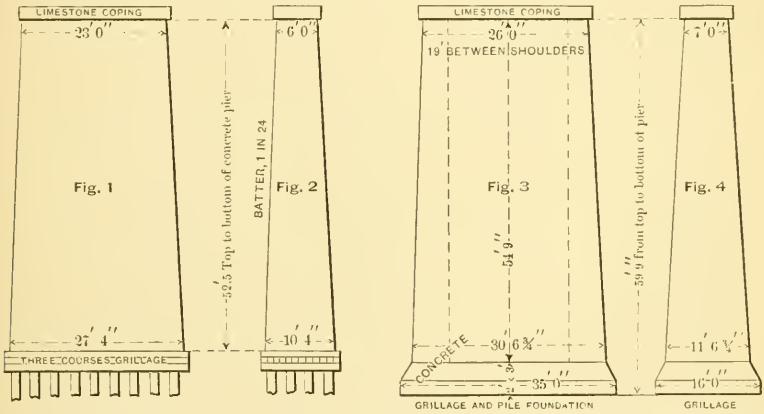
Mr. Gahagan. WALTER H. GAHAGAN, Assoc. M. Am. Soc. C. E.—In the correspondence on this paper Mr. Parsons says, "This is one of the largest pieces of pure concrete masonry used in railroad practice in the United States." The writer called the attention of George H. Pegram, M. Am. Soc. C. E., to this statement, and it was thought probable that much of the concrete work used for bridge piers in the South had not attracted attention, and that the majority of the members of the Society did not know to what extent concrete masonry had been used.

The bridge over the Arkansas River at Ft. Smith, Ark., has ten 200-ft. spans, and a 360-ft. draw span. The substructure for this work has thirteen concrete piers. This work was done under the direction of Mr. Pegram, the consulting engineer, as well as three other bridges, which were built in Louisiana in 1891, on the line of the Houston, Central Arkansas and Northern Railway. The writer was employed by the contractors, the Union Bridge Company, and had immediate charge of constructing the substructure for the Little River Bridge and the Red River Bridge, the latter crossing the Red River at Alexandria, La. These bridges were on the line of the railway just mentioned.

At Little River there were two piers built of concrete in the main channel which rested upon grillages and pile foundations, and two shore piers, which were two pair of cylinder piers. One of the concrete piers was 6 ft. wide by 17 ft. long between the shoulders at the top and had semi-circular ends. The batter was 1 in 24. The pier measured 52.5 ft. from the top of the concrete to the grillage. This pier contained 379 cu. yds. of concrete masonry (see Figs. 1 and 2). The other pier is a pivot pier carrying a girder draw span of 148 ft. It is 10 ft. wide and 12 ft. long between shoulders, with no batter but an enlarged base similar to that shown in Figs. 3 and 4. This pier

measures 51.9 ft. from the top of the concrete to the top of the grillage and contains 390.4 cu. yds. of concrete masonry. Mr. Gahagan.

The bridge across the Red River has six concrete masonry piers, all founded upon the natural ledge of rock. Three of these piers are 6 ft. wide at top by 19 ft. between shoulders, with semi-circular ends. Two of them, the draw rest piers, are 7 ft. wide by 19 ft. between shoulders at their tops, with semi-circular ends. The sixth is a pivot pier, a vertical shaft 27 ft. in diameter. The batter on these piers, excepting the pivot pier, is 1 in 24. The Red River piers measured from 44 to 53 ft. from the top of the concrete to the foundation. The quantity of concrete in them was 2 827 cu. yds.



At the same time they were under construction, the Union Bridge Company was building for the same company a bridge across the Ouachita River. This has five piers, two of them, the shore piers, being cylinders dredged down and filled with concrete. The channel piers are of concrete masonry resting upon grillages and pile foundations. The center pier is a pivot pier and carries a 360-ft. draw span. This pier is a circular shaft 27 ft. in diameter and measures 52.25 ft. from the top of the concrete to the top of the grillage. This pier contains 1 153.8 cu. yds. of concrete masonry. The draw rest piers are 7 ft. wide by 19 ft. between shoulders at the top of the pier. The batter is 1 in 24. These two piers contain 583.5 cu. yds. of concrete masonry each, and measure 59.75 ft. from the top of the concrete to the top of the grillage. The three piers contain 2 320.8 cu. yds. of concrete.

The total amount of concrete masonry in the piers for these three bridges is as follows:

Ouachita River crossing	2 320.8	cu. yds.
Little River "	769.5	"
Red River "	2 827.0	"
Total	5 917.3	"

Mr. Gahagan. The concrete for the Ouachita and Little River Bridges was mixed with a machine. That for the Red River crossing was mixed by hand in very much the same manner as described by the author, except the dry sand was first spread upon the platform, the cement spread evenly over the sand, and the cement and sand were then turned until they were thoroughly mixed dry. Water was next added and a mortar made. This mortar, when properly mixed, was spread out, and the broken stone thrown into it and the whole mass turned again two times before being put into the piers. The proportions used in mixing the concrete were 1 part of Portland cement, 2 parts of sand and 6 parts of broken stone. Figs. 3 and 4 show the long slender shafts, and certainly indicate that the designers and constructors of this work had no doubt but that concrete masonry was the only proper thing to use under certain conditions.

In 1892, on the Chicago, Rock Island and Pacific Railway extension through the Indian Territory, a bridge was built over the Red River near Doss, Tex., having piers entirely of concrete, no coping stone being used. The amount of concrete used in this work was over 3 000 cu. yds. The writer is at this time engineer and superintendent for the contractors who are building concrete masonry piers for a railroad bridge across the Arkansas River. More than 3 000 cu. yds of concrete masonry will be used. The proportions are 1 part of Portland cement, 2½ parts of sand, and 5 parts of broken stone.

Mr. Tuska. G. R. Tuska, Jun. Am. Soc. C. E.—Mr. Abbot has suggested a method of obtaining a smooth surface "by placing the mortar along the sides of the mold." At the Lonesome Valley Viaduct, where the work consisted mainly of small piers with a large amount of exposed surface, this method would require so much more cement than by the method followed as to make a material difference in the cost, with no advantage over the surface obtained by "cutting the sides" as explained in the paper. There is also the disadvantage of the piers not being homogeneous throughout, which might cause the concrete to crack from the different rates in expansion. The method of plastering the sides after the concrete has partially set is a poor one and not to be recommended, as in every case the author has seen it has caused the surface to break away soon after setting, on account of the differences in the rate of expansion of the materials.

As to the effect of frost on the structures. It is now a year since these masses of concrete were made. They have gone through hot dry weather and also freezing weather since erection, but no bad effects have been noticed. In fact, not even the sharp edges of the pedestals, etc., have been broken off. All the masses of concrete are in as good condition as when first erected, and no trouble of any kind has been experienced since completion.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

762.

Vol. XXXIV.—October, 1895.

TOPOGRAPHY ON THE SURVEY OF THE MEXICO-UNITED STATES BOUNDARY.

By J. L. VAN ORNUM, Assoc. M. Am. Soc. C. E.

READ AT THE ANNUAL CONVENTION, JUNE, 1895.

WITH DISCUSSION.

The adaptation of plans to the purpose in view constitutes the aim of the civil engineer. However general in scope a method may be, it is modified in characteristic details by ever-varying requirements of natural conditions, of the project, and of the necessary cost. The appreciation of the conditions involved characterizes the successful project, whether in the survey, design or construction of engineering work.

It is for the purpose of showing the adaptability (both in their usual form and in their elasticity to suit various conditions) of well-known surveying methods, and especially of the use of the stadia, that this paper is written. In discussing the survey there will be given a general review of the methods adopted under the various conditions, passing lightly over such details as are of general knowledge and use

at the present day and amplifying more those methods and details which are, perhaps, somewhat new to engineering literature and to surveys as at present conducted.

This boundary survey was made under the direction of two commissions, one representing Mexico, and the other the United States. Each commission made a complete survey, with independent parties, for its own nation. The commissioners for the United States were Lieut.-Col. J. W. Barlow and First Lieut. D. D. Gaillard of the Corps of Engineers, U. S. A., and A. T. Mosman, M. Am. Soc. C. E., of the United States Coast and Geodetic Survey. The field work of the survey was begun in February, 1892, and was finished in October, 1893. During this time the author had charge of the topography under the United States Commissioners.

The survey included all the frontier between the two republics, except the river boundaries, thus excluding that of the Rio Grande from El Paso, Tex., to the Gulf of Mexico and that of about 24 miles along the Colorado River below Yuma, Ariz. The overland boundary as surveyed consists of five segments, as provided in the Gadsden treaty of 1853, starting at the intersection of the Rio Grande and the parallel 31° 47′ north, and running west along this parallel 100 miles; thence due south to parallel 31° 20′ north; thence west along this parallel to the 111th meridian west of Greenwich; thence in a straight line to the Colorado River at a point 20 miles below the mouth of the Gila River; and (the fifth segment) from the point where the Gila River enters the Colorado River in a straight line to a point on the Pacific Ocean 1 marine league south of the south end of San Diego Bay. The last two segments were, then, on neither parallels nor meridians.

This boundary was marked soon after its establishment with as great care as was practicable at that time and in a desert country far from communication and supplies. Fifty-three more or less permanent monuments were erected in the distance of about 675 miles, but many became scattered and lost, so that there were eleven gaps between monuments of about 20 miles and more, and one space of over 100 miles. In consequence there were disputes concerning ownership of property and customs difficulties, and after the lapse of nearly 40 years a second treaty was formed between the two republics, providing for a resurvey based upon the previous one (making authenticated monuments of that survey fixed points of the boundary to be marked again), and provid-

ing that distances between monuments should never exceed 5 miles. In pursuance of this treaty and as determined by the resurvey, permanent monuments of cast-iron have been erected and the old authenticated monuments have been rebuilt of dressed stone in such a way that throughout the boundary the 258 monuments are visible one from another.

The nature of the country to be traversed had a most important bearing upon the organization of the parties and the methods adopted for the survey. Little was known of extensive portions of the boundary, and there was much difficulty in securing definite information concerning it. The fact that about as accurate a map of this region as had ever been made was published nearly two centuries ago (1701) by Father Kino, a Jesuit priest, will indicate the paucity of accurate information. Advantage was taken of the reports on Major Emory's survey (1849–1855), of a reconnaissance by Captain Symons about 10 years ago, and of such other evidence as could be secured. General plans were thus formed, and were modified as found necessary by reconnaissances varying from 50 to 150 miles in advance of the working parties.

The whole region is essentially arid, some portions more so than others, but all natural features and conditions are such as arise from its arid character. The country consists of a succession of plains and mountain chains, the latter extending generally from northwest to southeast and varying in width from 2 to 20 miles. The plains are flat, sandy wastes, rarely showing any irregularity or erosion; the cactus and sage brush form their distinctive and often only vegetation. The mountains vary in height from a few hundred to 4 000 ft. above the plains and are generally igneous or metamorphic in composition. Volcanic peaks are frequent, and at times the lava has poured in a great flood over the plains, covering them for miles with a sheet of liquid fire from a few inches to several feet in depth, and making a desolate region even more desolate and forbidding and furnace-like, as the lava rock catches and throws out again the fierce heat of an unclouded sun. Mountain vegetation here is also arid in character, the cactus being the prominent feature. The hardy and stunted ironwood, manzanita and evergreen oak are found on different ranges, and, very rarely, pine, sycamore and other trees, wherever they can find enough moisture.

The extremely precipitous character of the mountains is remarkable. Vertical angles of 40°, 50° and 60° were often necessary, and cliffs were encountered that could not be scaled; at times it was necessary to occupy with transits pinnacles hardly large enough to admit of the extended tripod legs. As a general illustration, at one point of the boundary a sight of about 2 000 ft. was taken from one mountain ridge to the next, between which there was a ravine about 1 000 ft. in depth. Before 10 o'clock the transitman began his descent from the ridge to cross the ravine and to climb to the point just observed, while he signaled the stadiaman to cross from that point a second ravine to a third ridge for obtaining an advance sight. Approaching sunset forced them both to return to camp before they could reach the intended points. The larger part of the day was taken in the descent and partial ascent, which suggests the rugged steepness of the mountains and the great care necessary to escape disaster. Similar conditions were frequently encountered. The greatest difficulty in this arid country is to secure water. Springs, wells, lagoons and streams were made use of as found, and twice great natural rock basins in the mountains, which caught rain-water and held it long after the rainy season, gave a very opportune supply. Still it was frequently necessary to draw the water 15 or 20 miles to camps, sometimes 50 miles, and in one case over 100 miles. The lack of water and its poor quality often caused inconvenience, but never suffering from its failure, though sometimes the margin of safety was small.

The summer temperatures are very severe; not so trying, however, in the plateau region of the eastern half as in the low-lying desert region on both sides of the Colorado River. Though from design the worst portions of the deserts were passed in the spring, during June and July of the second summer the parties that crossed the Yuma Desert were subjected to a temperature rarely falling as low as 90° through the night, and reaching at least 104° for four or five hours nearly every day, and on one day indicating 118° in the shade. The standardized thermometer would read only 154°, so that the temperature in the sun could not be determined.

The dryness of the atmosphere tempered the great heat in its effects, else it would have been unendurable. The only shade was furnished by the tents, vegetation affording none. Not only were the engineers working constantly in the sun, but, unless within reach of the camp, the noonday rest found no cooling shade, and proved, rather, a hardship.

Temperature, dry atmosphere and rugged mountain ascents all conspired to tax endurance and to necessitate a bountiful supply of water. Men would drink 2 galls. or more a day, suffering no ill effects. The country was, in the main, uninhabited. The Southern Pacific Railroad was generally the base of supplies. This necessitated a haulage, for varying distances up to 80 and 100 miles, of provisions for the engineers and the necessary escort of troops, and of forage for the animals.

Such, then, were the principal obstacles and hardships. They necessitated as small parties as possible, and efficient organization and methods.

The determination of each of the three right-line sections of the boundary, the meridian portion and the two western sections, consisted in the prolongation of the line from its determined direction at one end. The tracing of the two curved-line sections involved frequent astronomical stations. These were established at successive distances of about 20 miles. Observations were made to determine latitude and azimuth, and tangents to the parallel at each astronomical station were laid out and prolonged to the next station. Intermediate points on the boundary were determined by computed offsets from the tangent. A description of methods used for this work does not properly come within the scope of this paper. Descriptions of the latitude and azimuth determinations and of methods used in tracing long straight lines in the field are fully given in the *Transactions* of the Association of Engineers of Cornell University, 1894, by Mr. John F. Hayford, who had charge of this work on the survey.* These tangents and right-line portions of the boundary, as astronomically determined and prolonged, formed the basis for the topographical survey, and the work of the topographical parties begins here.

The whole boundary was measured by the stadia. At the first it was questioned whether the chain or stadia would give the more accurate results, although there was no doubt that in hilly and mountainous regions the stadia would be more accurate and very much more expeditious. Accordingly both were tried together in a country particularly favorable to the chain because of its generally flat and treeless character. Over a portion of the same line, Señors Molina and Contreras determined distances by a well-planned triangulation for the Mexican government in 1855. Corresponding distances as found by stadia and chain are given in the table, distances to follow being always in meters, the unit used also on this last survey.

* E. L. Ingram, Assoc. M. Am. Soc. C. E., had charge of about two thirds of this "line" work of the survey

TABLE OF DISTANCES AS DETERMINED BY STADIA, TRIANGULATION AND CHAIN.

Error.	Stadia.	Triangulation.	Corrected chain.	Error.
$-\frac{1}{600}$	13 171	13 193	13 193	0
$+\frac{1}{19969}$	19 970	19 969	19 965	$-\frac{1}{4992}$
$-\frac{1}{706}$	21 473	21 500	21 474	$-\frac{1}{827}$
$-\frac{1}{858}$	21 598	21 629	21 605	$-\frac{1}{901}$
$-\frac{1}{3803}$	22 813	22 819	22 804	$-\frac{1}{1521}$
$+\frac{1}{639}$	49 957	49 879		
$-\frac{1}{21284}$	148 982	148 989	= Total distances.	

The total ratio of error between triangulation and chain is $-\frac{1}{1436}$ from the table; between triangulation and stadia for the same distance, $-\frac{1}{1166}$; and for the total continuous distance, $-\frac{1}{21284}$.

It will be noticed that over the chain-measured distances the heading is "corrected chain"; this is so stated because at different times the stadia detected the dropping or addition of a chain length in the chaining record. This occurred six times in the above distance, and, when detected, the section in question was remeasured. The error was invariably found to be by the chain and its record thus corrected, otherwise there would have been six errors of chain measurement of 20 m. each. The stadia places all the responsibility for correct measurement on the trained transitman, and so reduces the danger of systematic errors to a minimum. These considerations indicated the greater reliability of the stadia measurement, even in the level country.

During the progress of the survey here described, some other sections were triangulated. All distances so determined are compared with the corresponding stadia distances in the following table:

TABLE OF DISTANCES AS DETERMINED BY TRIANGULATION AND STADIA.

Triangulation.	Stadia.	Error.
223.6	222.7	$-\frac{1}{248}$
411.7	412.7	$+\frac{1}{412}$
757.6	755.0	$-\frac{1}{291}$
785.0	785.3	$+\frac{1}{2617}$
943.8	945.5	$+\frac{1}{555}$
983.3	985.2	$+\frac{1}{518}$
1 273.2	1 272.8	$-\frac{1}{3183}$
1 346.6	1 349.0	$+\frac{1}{561}$
1 788.0	1 789.3	$+\frac{1}{1375}$
4 433.0	4 446.0	$+\frac{1}{341}$
73 095.5	73 056.6	$-\frac{1}{1879}$
86 041.3	86 020.1	$-\frac{1}{4059}$

In considering this table it should be borne in mind that the first ten distances were measured under conditions as severe as are ever likely to occur in surface measurement. The region was a succession of ridges and deep cañons often on too large a scale to admit of sighting from one ridge to the next, and so necessitating large vertical angles; and at times the vertical angle for a reasonable sight would be so great that readings of excessive length would be taken in preference. Its rough, abnormal character may be seen from the fact that it would certainly have taken two or three weeks to chain the distance of about 8 miles, and even then the result would have been very questionable, owing to the error and confusion from incessant "breaking chain." The measurement by stadia occupied about 20 hours.

A third portion of the boundary has been triangulated since the completion of the regular survey. This was done because here the Mexican and American stadia distances showed a considerable disagreement, and so it is reasonable to suppose that it furnishes a check on the poorest, and not the average, stadia work. The comparison of distances with the original stadia measurement is here given by sections:

Table of Distances as Determined by Triangulation and Stadia.

Triangulation.	Stadia.	Error.
257	256	$-\frac{1}{257}$
1 528	1 530	$+\frac{1}{764}$
2 471	2 475	$+\frac{1}{618}$
3 074	3 075	$+\frac{1}{3074}$
3 308	3 310	$+\frac{1}{1654}$
3 324	3 333	$+\frac{1}{369}$
3 334	3 340	$+\frac{1}{835}$
3 475	3 478	$+\frac{1}{1158}$
3 624	3 628	$+\frac{1}{906}$
4 012	4 012	0
5 694	5 699	$+\frac{1}{1139}$
5 776	5 794	$+\frac{1}{321}$
5 987	5 992	$+\frac{1}{1197}$
6 061	6 068	$+\frac{1}{866}$
6 685	6 698	$+\frac{1}{514}$
58 610	58 688	$+\frac{1}{751}$

Each of the three tables gives the errors of each component section and its total ratio of error. In all, 293 640 m. (182½ miles) of stadia-measured boundary have been triangulated, in which the total difference in length is + 50 m., or an error of $\frac{1}{5873}$. However, this method of

stating the accuracy by aggregate lengths, though often used, is not the truest or fairest ; such ratio of error should rather be a mean of all the different errors of closure of each short portion checked, giving that of each section its relative weight. The error is $\frac{1}{9\,4\,9}$ when so considered.

The topography taken formed a belt 4 000 m. ($2\frac{1}{2}$ miles) wide for the whole length of the boundary and covered an area of about 1 700 sq. miles on the American side. The work as planned involved the instrumental location and determination of the elevation of all surface irregularities and the position of roads, trails, towns, ranches, wood, grass, and water. All such topographical points were taken as would appear on the maps drawn to the scale of $\frac{1}{3\,0\,0\,0\,0}$, and with contours 20 m. apart.

The natural features of the country to be traversed and the conditions to be encountered plainly indicated the transit and stadia as the best field equipment. The necessity of reducing the size of the expedition to its lowest limit suggested the plotting of the maps away from the field. The topography, embracing a belt of country 4 000 m. ($2\frac{1}{2}$ miles) in width, required a general onward movement and a concentration of all instrument men and instruments on topography as mountain ranges were encountered, all working to advantage in limited areas. The occasional remoteness of the work from available camps, the very rough and difficult nature of many regions encountered, and the necessity of utilizing even the many very windy days called for the lightest and most compact equipment possible, while the precipitous character of many of the mountain ranges, requiring vertical angles of from 30° to 65°, frequently precluding the possibility of resection, and, at times, offering peaks for instrument stations that would barely furnish room for the tripod, confirmed the choice of the transit as the only practicable field instrument.

The party that measured the tangents was also charged with securing the topography adjacent to and within, perhaps, 200 m. on both sides of the boundary with greater detail than was used elsewhere in the topographical belt. A level line was run with exceptional care along the boundary to secure its profile; the elevations so secured also furnished elevations for the general topography. The tangent lines thus furnished in their directions, distances and elevations the basis of the topographical lines.

Where necessary and advantageous, transit lines would start at a point on a tangent line, extend in a general northward direction to near the northern edge of the 4000 m. zone, and turn again to the tangent line at another point, thus securing a check on their accuracy of azimuth, distance and elevation. The general course of procedure corresponded, in the main, with the usual practice in stadia surveys. Azimuths were carried continuously and measured the deviation from the meridian, or, more strictly, from a line at right angles to the tangent. As this involved two sights over each course to secure the azimuth, readings for distance and vertical angle were also taken on both foresight and backsight, giving a double determination on each course of its length and change in elevation. Topographical points were then located by side sights from these main-line stations, or, at times, from transit stations on unclosed spur lines. At times, when difficult mountain ranges were encountered, the parties engaged in measuring the tangents and carrying forward the level line were withdrawn from their usual work to reinforce the others, all parties then working advantageously together on pure topography. Where such concentration was not necessary, each party would carry on its own particular work. Nearly three-fourths of the total area was covered by this general method.

About 50 sq. miles was taken by a method of intersections. In the region so covered there were a few commanding peaks, below which lay the terrene in all its irregularity. The general procedure here consisted in the establishment of station signals on these commanding peaks, and from numerous stations on the boundary the tangent transitmen would take azimuths and vertical angles to these signals, from which the location and elevation of each station were computed; then these same stations were occupied, two at a time, in such order as to give the best intersections on points to be located. One engineer with a flagman, both mounted, where the country would permit, would traverse an advantageous area; the flagman signaled at each necessary point of the area, and the engineer made a contour sketch of the terrene, showing the position of the points from which the signals were made. At the same time the two transitmen at their stations would watch the flagman, and, on a signal from him, each would take the azimuth and vertical angle to his signal, held vertically on the ground. The engineer and the two transitmen would all note the time of each

sight, to have a check distinguishing corresponding observations. The similarity to the intersection method of locating soundings in hydrographic surveys is apparent; points are located in the same way, the difference being that on this topography the elevation of each point is also determined. Computation from the vertical angle of each transit applied to the scaled or computed distance gives two independent determinations of elevation.

While topography taken in this way is not capable of the accuracy of the usual method, it is still at times advantageous, and was used to facilitate and hasten the field work. Eighty per cent. of the points so located checked by the two computations within less that 3 m. in elevation, even when the sights were 10 000 or 12 000 m. in length; of those not so agreeing, that which was the true elevation was indicated by reference to the contoured sketch.

It was hoped in such intersection work that aneroid barometers might be used for obtaining the elevation. To test their accuracy they were used in the approved way, one being stationary in the field to note fluctuations in the atmosphere itself, while two, for comparison, were carried and read at each located point. Comparison of all three was made before and after work, and care used to secure good results. However, the test confirmed the author's opinion that they are useful only as reconnaissance or approximate instruments. The two carried and read simultaneously would sometimes differ 20 m., and elevations computed by comparison with the stationary barometer readings would at times differ from the instrumentally determined elevations by as much as 40 m. In this connection it may be of further interest to state that the mercurial barometer carried on the survey of 1849–1855 for determining elevations gave often very erroneous results, varying by different amounts from the true elevations to an extreme error of more than 400 m.

The stress of adverse natural conditions in the most inaccessible and forbidding portion of the boundary made necessary a further modification in the topography for an area of about 450 sq. miles. The region so covered combines obstacles to its penetration to such a degree that where traversed by the overland route for emigrants from Mexico to California, scores and probably hundreds died of thirst on the road. A portion of this region was considered absolutely impossible of penetration, and no expedition had traversed it. The sur-

vey of 1849–1855 attempted it twice and failed. In the face of this record, and with the summer heat of the desert approaching, it was considered necessary to expedite the work in every way possible, and the result proved the wisdom of the plan. With the increased celerity of progress due to the modified plan, the expedition was barely able to traverse this region and complete the work without delay and, perhaps, disaster.

The method used here was unchanged for the measurement and profile of the line and for the topography within 1 000 m. of the boundary. The modified plan affected only the belt from the 1 000 m. to the 4 000 m. distance, covering a zone 3 000 m. wide for the region in question. This belt was to be so taken as to show on the maps all natural features that could be delineated by hachures instead of by contours at the usual scale, $\frac{1}{3000}$, thus showing all the main characteristics, but necessitating a less refinement of work. The procedure consisted in locating and obtaining the elevation of flags placed on all prominent peaks and ridges from 1 000 to 4 000 m. from the line, by the transitman measuring the boundary. Another transitman would take azimuths and vertical angles from each of these located points on the other points in sight, thus covering the zone with a network of location lines. At the same time, he would take resections or stadia readings on the more important secondary features of the terrene, and complete the work by sketches. This method gave the instrumental location and elevation of the characteristic and principal natural features of the region, with approximation to the less prominent ones.

With these exceptions, the topography was secured, as before stated, by stadia location. In this work the average length of primary sights on the transit lines was 358 m., and that of the secondary sights, for general location and elevation, was 346 m. The average error of azimuth on the closed lines was 23″ per course. The total length of the 118 closed lines (not including the sections of tangents checked by triangulation and already given, and also excluding the section of tangent in each case furnishing the data for completing the closure) aggregated 826 516 m. (513½ miles). The average error of closure on these lines was $\frac{1}{752}$. The total change in elevation, or rise and fall, on these lines was 38 091 m. (124 970 ft.); the average error in closing of elevations was $\frac{1}{374}$ of the vertical component or change in elevation.

In considering these different results it should be borne in mind that the work was done with no effort to make a "record." On the contrary, the conditions were quite adverse, especially on account of the frequent excessive vertical angles, the vibration or boiling of the air, and the necessity of constant progress requiring work even in the numerous wind storms. It was the rule that stadia work should continue as long as it was a physical possibility for the stadia men to hold up the stadia rods against the wind. Elevations were carried on the closed transit lines from a level station at the beginning to another at the end. A discussion of the computation from one station to the next will illustrate the essential characteristics of all.

Consider the transit at a station of known elevation, taking an observation on the next in course. The interval is read, where convenient, on the stadia rod, the vertical angle is read for precisely this inclination, and the reading on the stadia rod of the middle cross-hair (for this same angle) is noted. The vertical component of the distance read, with its proper sign, plus for elevation and minus for depression angles, minus the recorded reading of the middle cross-hair, gives the quantity to be applied to the elevation of the instrument at the occupied station, to secure the elevation of the advance station, curvature and refraction being disregarded.

It is true that in stadia surveys it is often the practice to read the distance on a convenient part of the stadia rod, and then to change the telescope slightly before reading the vertical angle, until the middle cross-hair stands on the stadia rod at a height equal to the height of instrument; the purpose is to carry forward the elevations by simple distance-into-angle computation. This practice introduces error into even the horizontal (distance) component; while in the vertical (elevation) component, it introduces error varying with the space on the rod which the middle hair passes over during the change of the telescope. The other alternative, viz., to read the distance with the middle cross-hair pointing at the height of the instrument point of the stadia rod, would often be even more annoying, as it would restrict the length of sight to about 300 m. It is frequently necessary to take longer sights than this; it will be remembered that the average length of sight on the boundary survey was greater. Again, as the disturbing effect of refraction increases enormously toward the ground, as will be seen in the discussion of refraction following, it is not advisable to set the

lower hair within less than a meter (about 3¼ ft.) of the ground, even if the foot of the rod is unobstructed. It is true that if these practices are ruled out it necessitates the division of the stadia rod into true units, instead of fitting such divisions to the transit interval. This phase of the case is discussed later in the paper.

When the advance station is occupied, a similar process gives another determination of the station's elevation, and a combination of the two eliminates curvature and refraction as well as errors due to the instrument being in imperfect adjustment or due to the effects of heat upon it.* Thus, when no error is made in observation, and refraction remains constant from the time of foresight to the time of backsight, the equation

$$x = e + \tfrac{1}{2}\,(a + m + n - b)$$

will give the true elevation of the advance station, where

e is the elevation of the known station,

a is the height of the instrument at the known station,

m is the total vertical component of the foresight,

x is the elevation of the unknown station,

b is the height of the instrument at the unknown station, and

n is the total vertical component of the backsight.

The average error in closing before the adoption of this formula was more than a half greater than after its use.

For the purpose of determining the error of closure in elevation relative to the degree of vertical angle, the author grouped the lines according to the size of those angles and plotted the corresponding errors as ordinates. The equation of the curve so formed, where α represents the average vertical angle in degrees is, for meters per kilometer,

$$\text{Error} = \frac{30\,(\alpha\,.\,\text{tan.}\ \alpha) + 8}{100}$$

or, expressed in feet per mile,

$$\text{Error} = \frac{30\,(\alpha\,.\,\text{tan.}\ \alpha) + 8}{19}$$

It would seem that the error should vary about as the tangent of the angle only. The presence of α itself in the equation must be due to the fact that in practice, stadiamen will not hold the stadia rod

* Discussed by the author in the *Journal* of the Association of Engineering Societies, August, 1893.

exactly vertical, even with the aid of a plumb line, the tendency being to hold it perpendicular to the line of sight, and an error varying with its inclination is thus introduced.

More favorable conditions in the field would change the constants of the equation and give a smaller probable error. Considered with reference to results with wye levels, this error is large, though not so surprising when the rise and fall of the lines is remembered. It was greater than it should have been because the reduction formula before given was not devised until a part of the survey had been completed, and because work was often continued when wind storms rendered it impossible for the stadia rods to be held steadily for the reading. A greatly increased accuracy could be readily secured if the vertical circles of transits were made to read to fractions of a minute, if the telescope bubble-tube were made and ground with as much care and refinement as that of a wye level, if a bubble-tube were attached directly to the vertical circle vernier, as has been done on the Lake Survey and Mississippi River Survey, instead of trusting to the plate bubble, and if stadia hairs are made as fine as possible consistent with distinctness.

Here the author would again call to mind the fact that on the boundary survey the usual stadia methods were generally followed. Some modifications were found necessary because of unusually severe conditions of work and a description of them follows; these refinements were employed to secure more accurate results where such conditions would have unduly magnified the smaller errors that generally compensate in time, and so are usually disregarded. The desirability of applying the following refinements must be decided by the circumstances of each case.

In determining the interval factor of a transit's stadia hairs, whether for reduction or for marking the stadia rods correspondingly, especial care should be used to secure as close an approximation to average working conditions as practicable. This includes a consideration of the time of day and the elevation of the line of sight above the ground to involve the average amount of refraction; the length of sight, varying from the shortest to the longest usual length; frequent tests of the interval; readings by the habitual user of the transit to involve his personal equation; and great care in the measurement of the base for the determination, numerous readings at the different distances, frequent determinations and carefulness in all the details.

The effect of differential refraction on the determination is well stated in a paper by Mr. L. S. Smith, of the University of Wisconsin, and published by that University in its engineering *Bulletin*, Vol. I, No. 5. Mr. Smith shows from extended experiments that the amount of refraction of the line of sight of the upper stadia hair usually differs from that of the lower one. Sometimes this differential refraction increases the intercept and at other times diminishes it. Early in the morning, during the summer in Wisconsin, and up to about 8.30 A. M., the effect of refraction on a stadia reading is to increase the intercept, and so the distance beyond the average reading. From that time to about 2.30 P. M. the intercept is less than the mean, and from this time until night the intercept again becomes too great, as in the early morning. Fig. 1 shows a typical case. It illustrates how

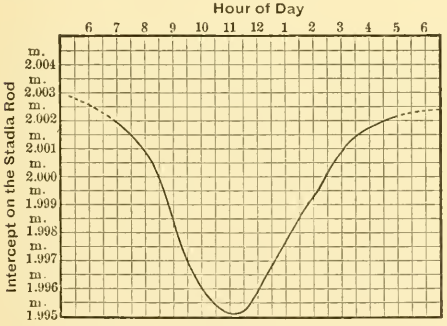


FIG. 1.

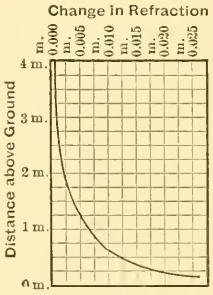


FIG. 2.

differential refraction, though slight in amount, does and must affect the intercept. The curve must be taken merely as suggestive, for its ordinates vary with the state of the atmosphere and ground, temperatures, weather, latitude and change of the seasons. A variation of these conditions appears to affect the hours and amounts of the maximum and mean distortions, and perhaps the direction of curvature itself, while not changing the fact that at certain times of day refraction is abnormally great and at others abnormally small. This principle last expressed is the one affecting stadia intercepts, and should have great weight in fixing the proper time and method of interval determination. Mr. Smith also found that within 1 m. of the ground the effect of refraction is greatly increased, resulting in increasingly distorted intercepts as the ground is approached. Fig. 2

illustrates a typical case clearly, showing the increased error nearer the ground, which seems to occur always.

The stadia interval should, then, be determined when the lower cross-hair does not come within 3 ft. of the ground, and should involve conditions which will secure the average intercept. These conditions are so various and the amount of differential refraction at any time so uncertain that it becomes a necessity to determine an interval through all the hours of day which will constitute the working hours. Nor should a single determination be accepted as final, for, with all care in choice of conditions, unusual circumstances or changed conditions might be present to affect the result. No one should plan an extensive triangulation from a base measured only once. Determinations should be made at reasonable intervals of time with equal care, the mean of which is much more likely to secure average conditions, and so a truer value.

Neglect in regard to securing average conditions of refraction have proved, unwittingly, very troublesome to those who have not considered it. The author calls to mind an extensive and costly topographical survey where the interval was determined from a single series of about 10 sights taken at about 11 A. M., and used for several years as so obtained. From Mr. Smith's investigations it is seen that this hour is one of the most unfavorable for securing average conditions of refraction, and will introduce error in the unit of measure itself. In this case it would make the survey distances too great. This is in fact what did occur, the engineer in charge noting the resulting errors without appreciating their true significance. The use of a transit for months and years without again testing its interval is a mistake, both because an increased number of determinations gives greater reliability to the mean, and because changes in the cross-hairs, though rare, do sometimes occur. The author has the record of an interval which changed gradually, though not uniformly, more than 2% in a year. Engineers should be sure of their instruments. In leaving this subject, the importance of intelligent and careful detail should be emphasized, as it is the basis of all stadia work.

The consideration of focal distance plus distance from the objective to the center of instrument, $f + c$, is often discussed. By an optical law applied to the construction of the transit telescope this quantity, greater than 1 ft., must enter to give correct results. There is no reason why the law of optics and sound theory should be disregarded,

and it is the usual custom either to add $f + c$ to the sight or to so determine the interval factor that by omitting this $f + c$ distance from the base measurement, the $f + c$ is eliminated for a mean distance. This latter procedure, however, gives erroneous readings for distances greater or less than the mean length of sight fixed upon, and therefore should not be used for the more accurate stadia work.

The author believes that the best practice (in the interval determination) is for the transit to be set at a distance equal to $f + c$ back from the beginning of the base and then to add $f + c$ to each distance read. By using $f + c$ in this straightforward way, an increased accuracy is given to all the work, and especially to the measurement of comparatively short distances, where the ratio of error is at best always greatest and where there is need of care. An exception to either of these usages occurs in the article in the *Journal* of Association of Engineering Societies, January, 1893, on the survey of St. Louis, the writer of which advocated the determination of the interval in such a way that $f + c$ entered into the base measurement and then in practice ignored that quantity. The stated reasons for neglecting $f + c$ was that in the computation of the lines that writer found the distances were almost invariably read too great. However, this is not a necessary error of stadia work. Other surveys do not find these constant cumulative plus errors in distance ; the tables of distances given previously in this article happen to have a greater aggregate length of line where the stadia distance was read too short than the total in which it was read too long. It seems that this illustrates again that it is bad practice to ignore sound principle. If an explanation of these constant plus errors of the St. Louis survey is sought, it will be found to occur, not as a retribution for not juggling with $f + c$, but in the probability that the determination of the stadia interval itself was made in such a way or at such a time that refraction crept in as the disturbing element.

To have the stadia rod divided into true unit lengths instead of into approximate units to fit its particular transit is desirable for many reasons. It makes impossible any error due to an accidental interchange of stadia rods among transits ; any stadia rod not especially constructed for the particular transit may be used in case of emergency or desirability, viz., level rods may be used for stadia readings whenever desirable, as advocated by J. B. Johnson, M. Am. Soc. C. E., for stadia railroad preliminary surveys, and by Mr. Benjamin S. Lyman; the

stadia rods may be used for level rods or for any direct measurement when desired; or stadia rods and stadiamen may be transferred from one party to another where conditions require, and no error results. Two considerations already mentioned make this division into true units almost a necessity; the one that the distance above the stake at which the middle hair stands when the vertical angle is measured may be read in true linear distance; and the second, that error in the stadia interval, occurring either from a change in the cross-hairs or from a subsequent determination indicating that previous determinations have not developed the exact and true factor, may be corrected without the complete change of the stadia rod divisions, which would, at best, consume several days of delay and careful work, and might be impracticable.

Stadia.	Distance.	Stadia.	Distance.	Stadia.	Distance.
.001	.10	0.6	64.10	2.8	294.66
.002	.21	0.7	74 58	2.9	305.14
.003	.31	0.8	85.06	3.0	315.62
.004	.42	0.9	95.54	3.1	326.10
.005	.52	1.0	106.02	3.2	336.58
.006	.63	1.1	116.50	3.3	347.06
.007	.73	1.2	126.98	3.4	357.54
.008	.84	1.3	137.46	3.5	368.02
.009	.94	1.4	147.94	3.6	378.50
.01	1.05	1.5	158.42	3.7	388.98
.02	2.10	1.6	168 90	3.8	399.46
.03	3.14	1.7	179.38	3.9	409.94
.04	4.19	1.8	189 86	4.0	420.42
.05	5.24	1.9	200.34	4.1	430.90
.06	6.29	2.0	210.82	4.2	441.38
.07	7.34	2.1	221.30	4.3	451.86
.08	8.38	2.2	231.78	4.4	462.34
.09	9.43	2.3	242.26	4.5	472.82
0.1	11.70	2 4	252.74	4.6	483.30
0.2	22.18	2.5	263.22	4.7	493.78
0.3	32.66	2.6	273.70	4.8	504.26
0.4	43.14	2.7	284.18	4.9	514.74
0.5	53.62				

The division of the stadia rod to fit the transit is widely used and advocated. The reason for this is found in the fact that the observation gives the distance if $f + c$ is disregarded in the interval determination; or it gives the true distance by the simple addition of $f + c$ to the reading where the interval is correspondingly determined; while a true unit division is supposed to involve for each sight a complex reduction, taking both time and care. For example, if a transit has the stadia hairs so spaced that at a distance of 100 ft. the intercept is 0.9542 ft., or, in other words, its interval factor is 0.009542, it is generally supposed necessary to divide each reading taken on a true unit stadia rod

by 0.009542, and then add $f + c$, entailing a great amount of work. However, that supposition is not true. On the boundary survey, Mr. P. D. Cunningham devised a method whereby the true distance is derived with no more computation than is necessary when the untrue unit division of the stadia rod is made. The method is, in brief, as follows : A table is made (illustrated on the opposite page for the shorter distances with an interval factor of 0.009542 and a value of $f + c = 1.22$) giving readings for all distances from 0.1 up to the longest, in increments of 0.1. These are computed by dividing each distance wanted by the interval factor, and adding $f + c$, viz., for a rod reading of 3.4 the corresponding tabular distance is $\frac{3.4}{0.009542} + 1.22$. Similar computation is also made for distances from 0.01 to 0.09, differing from each other by 0.01, and for distances from 0.001 to 0.009, differing from each other by 0.001, in a way exactly similar to the above, except that $f + c$ is not added. Then the true distance corresponding to any stadia rod reading is found by a simple addition of quantities taken from the table. Thus, if the reading is 2.67, the true distance is 273.70, opposite 2.6, plus 7.34, opposite 0.07, which equals 281.04.

Fig. 3.

If a slide rule is accessible, each stadia rod reading can be divided by the interval factor and the corresponding true distance read from it, to which $f + c$ must be added; but this would involve more work than the use of a table. The table itself can be formed in half an hour by the aid of the slide rule, or in two or three hours without. By using such a table there occurs no more computation than is necessary when the rods are not divided into true units, and it permits all the advantages previously given for the correctly divided rod without occasioning any disadvantage.

The design of stadia rod used on the boundary survey is the metric one given in Fig. 3. The stadia rod with the foot divisions (Fig. 4) has been extensively used by the author on internal improvements and surveys. In

both Figs. 3 and 4, the hatched divisions represent red divisions on the rod. It is the general opinion that stadia rods should combine simplicity, clearness, distinctive divisions and subdivisions, and efficiency in indicating the shorter distances very accurately and in giving great distances readily. Those illustrated were designed with these characteristics in view, and their use confirmed their expected utility. With an erecting telescope at a distance of 250 m., the centimeter divisions could be seen and spaced by the eye into millimeter readings, while the decimeter divisions could be seen at a distance of 700 or 800 m., and correspondingly spaced into centimeters.

Fig. 4.

On one portion of the boundary more than 100 miles intervened between two successive points as marked in 1855. The line of sight between these points was obstructed by two mountain ranges, one 45 miles and the other 60 miles distant, and it became necessary to establish a station on the connecting line on the nearer range. This preliminary work was accomplished by successive approximations. Heliotropes were shown at the boundary points and a transit and a heliotrope were placed on each of the ranges. On one range a trial point was assumed and the transit on the other range lined in between it and the adjacent boundary heliotrope. When this was accomplished the transit was replaced by a heliotrope and the transitman on the first range lined in between that heliotrope and the boundary heliotrope adjacent to him. Repeating this operation time after time, a station was finally established on the nearer range. When on the regular survey this line so determined was prolonged with a theodolite and with all care, it was found that the established point was 0.87 m. from the true line, or had an angular error of 2".45. So close a result was exceptional, but it is given as indicating the reliability of ordinary transits and heliotropes when used with care and skill.

Such, then, are the methods, results and conclusions of this survey. They occurred under adverse circum-

stances necessitating boldness in length of sight, vertical angle, and in general details. Interpreted in this light, they indicate the results of probably extreme conditions, certainly not average ones. Its use and usefulness make the stadia system so extensively employed in continental Europe on topography and on surveys for railroads, canals and other public works deserve all the growing attention it is receiving, and more. In general economy, adaptability and efficiency it is unequaled.

DISCUSSION.

F. V. Abbot, M. Am. Soc. C. E.—The rivers in South Carolina are commonly bounded on both sides by extensive cypress swamps, and it is generally impossible to set up an instrument anywhere on the banks, which are overhung with brush. The mud is of indefinite depth. The speaker had a number of surveys made of these streams, using the stadia method and employing two boats. The transit was lashed firmly in the bow of one boat, so that it would not fall overboard. The men in the second boat tied a piece of cloth on a tree overhanging the water on the opposite side of the river and as far down stream as they could be seen from the transit boat, and held up a stadia rod directly against the rag. The rod was then read and the magnetic reading also taken from the transit boat, which then went down stream past the rod boat as far as it could and still secure a sight of the tree. From this new position a stadia reading and the magnetic bearing were taken of the rod held in the second boat at the rag. The rod boat then went down the river as far past the position occupied by the transit boat as it could be seen, and the distance and magnetic direction were read again. The transit boat was always on the left bank, and the rod boat on the right bank of the river. By repeating this process, it was possible to secure a very fair map of such a river, obtaining its width and length with more accuracy than in any other way with which the speaker was acquainted. Three men in each boat can survey 30 miles of river a day (down stream) without any great trouble if no soundings are made. Mr. Abbot.

W. L. Webb, Assoc. M. Am. Soc. C. E.—In using the stadia to measure distances between stations, the speaker had adopted a method of reducing the error in reading the rod with the upper and lower cross wires. After the reading with the three wires is taken, the tangent screw of the vertical motion is turned very slightly, just enough to give different readings. Theoretically a change in the vertical angle will produce a slight difference in the length of rod subtending the wire interval, but practically the difference is very small and may be Mr. Webb.

Mr. Webb. neglected. For example, with a vertical angle of 10° and a distance of 1 000 ft., the vertical angle would have to be changed 10′ in arc to make a difference of 1 ft. in the apparent horizontal distance and 10′ of arc in 1 000 ft. would mean a movement of 2.9 ft. of the wire over the rod. Entirely different readings are thus obtained and there is no tendency to repeat involuntarily an error made in the first set of observations. Any number of sets of readings may be taken in this manner, and the average will be much more reliable, especially for long sights, than a duplicate reading of the rod without any change in the vertical adjustment of the telescope.

CORRESPONDENCE.

Mr. Allen. KENNETH ALLEN, M. Am. Soc. C. E.—The prevailing opinion regarding the use of the stadia has been heretofore that, at best, it would answer as a rough expedient, comparable rather to pacing than to chaining as regards accuracy. Among those familiar with its use, however, its great value has been recognized and its adoption advocated where single errors of $\frac{1}{300}$ to $\frac{1}{1000}$ would be allowable. The details of the work may vary greatly, but from a considerable experience the writer wishes to emphasize his approval of two of the author's recommendations.

First.—The rod should be divided to true units, feet or meters, and decimals. Beside the reasons given in the paper, the writer has found it preferable to take the levels directly when practicable instead of by the vertical arc, which can only be done by means of such rods. If no means are at hand to provide a special rod a Philadelphia level rod will answer nearly as well for distances under 1 000 ft. in length.

Second.—Stadia hairs should be adjusted with $f + c$ added to the distance, and this amount added to all observed distances. This does away with the use of calculations or tables, and, so far as it goes, is correct.

But some rather recent surveys, including that described by the author, indicate that the stadia may be used for work of a much higher grade by making due allowance for refraction. The writer would also recommend for this purpose telescopes of higher magnifying power than those in ordinary use. The very great rapidity and economy attained by the use of the stadia make every increase in its adaptability a matter of importance, and the writer believes such an increase to follow from the carefully conducted investigations of the survey described.

The aneroid is believed to be deserving rather more credit than would follow from its use on this survey. The climatic conditions may have been peculiarly unfavorable. Large individual errors may occur under the most favorable circumstances, but with the precau-

tion of using several instruments and making the proper reductions, the writer is surprised at the discrepancies found. It has been his practice to use several aneroids as checks on one another, but the following errors, determined some four years ago in western Pennsylvania from observations at points of known elevation, were (with the exceptions noted) from the mean of two observations taken with a single aneroid. The extreme range of elevations being comprised between 865 and 1 339 ft. above sea level, the results are of course not comparable to mountain work, but serve as a sample of the accuracy of such determinations in ordinary hilly country. Mr. Allen.

Station...	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
	0*	−13	+6	−17*	+2*	−1	−2	+2	+5	−2*	+7	−8*	−19*	+12	−2

Station...	16	17	18	19	20	21	22	23	24	25	26	27	28	29
	+36*	+10	+5	+32	+19	0*	0	+11	+5	−16	−20	−19*	−12	0*

WILLIAM B. LANDRETH, M. Am. Soc. C. E.—The paper calls to the writer's mind some work done on a stadia survey in the states of Sinaloa and Durango, Mexico, in 1881 and 1882, by H. M. Wilson, M. Am. Soc. C. E., and the writer. Mr. Landreth.

Edward P. North, M. Am. Soc. C. E., then chief engineer of the Ferro Carril Sinaloa y Durango, in looking up the matter of a possible railroad route from the Pacific coast, at Altata, Mex., to the city of Durango, found that there were no reliable surveys or maps of the country between those places, and to obtain some information thereon, decided to have a survey made of the principal mountain trails between them. As the examination was to be of the nature of a reconnaissance, it was deemed advisable to use the stadia to reduce the cost and time of making the survey. Two routes were to be taken from Culiacan; a southern one, passing through the towns and villages of Las Tapias, Santa Cruz, Las Vegas, Cosala and Vivorillias to Durango; and a northern one, returning from that city through Canitlan, Guatimape, Santiago, Santa Catalina and Canelas to Culiacan. The two routes chosen were those followed by pack trains crossing the mountains, and would give a closed survey, affording a means of approximately checking the accuracy of the work.

Each party consisted of an engineer, who acted as transitman, and native rodmen, stake drivers, machete men for cutting brush, and a mule driver. Mules were used for carrying the provisions and clothes of the party, and no tents or outfits for permanent camps were provided, owing to the rapidity necessary in the work. A party in charge of the writer started on the southern route early in October, 1881, from the city of Culiacan, with instructions to follow the trails generally taken by pack trains between the villages, and to report progress whenever possible, by means of travelers or mule trains met on the road.

The transits used were made by Messrs. Heller and Brightly, with

* From single observations.

Mr. Landreth.

stadia hairs set to cover 1 ft. between upper and lower hairs on a Troy leveling rod when set 100 ft. plus a constant distance from the instrument. The constant added was frequently checked, but it was found that the stadia hairs needed little adjusting.

The rodman, accompanied by an experienced Indian stakeman, set wooden hubs in the angles of the trail, and after the readings were taken, left a small peeled sapling with a piece of white paper attached near the hub, so that the transitman could easily find it on coming up; he then went ahead to locate the next one. The transitman set up over the hub, turned back on the sapling at the last hub for a backsight, took the readings, placed the sapling behind the hub, and went on to the next one. This method dispensed with the services of a rear flagman, and it was determined by experiment, before adopting it, that none of the accuracy of the work was sacrificed by so doing. The readings included the angle of total deflection from a base line in Culiacan, magnetic bearing, reading of upper, middle and lower cross hairs, foresight with level when possible, vertical angle, a measurement of the height of the instrument from hub with tape, and aneroid and thermometer readings.

From Culiacan to Cosala the route lay among small foothills and crossed the valleys of one large and several small streams, and the thick underbrush made the progress slow. Soon after leaving Cosala, the narrow, rocky gorge of the San Lorenzo River was entered, and followed for 40 miles to the Sierra Madre Mountains at Vivorillias, where the trail rose 6 000 ft. in about 10 miles, to an elevation above the sea of 11 000 ft. From the summit a descent was made into long mountain valleys, which were followed for about 120 miles to the city of Durango. Durango is a place of about 30 000 inhabitants, lying in a wide plain at an elevation of 8 000 ft. above the sea. After a week's rest at Durango, writing a report of the trip, repairing the camp outfit, and giving the pack animals a chance to recover flesh and vitality, a start was made on the northern route December 26th. For the first 120 miles, to Santa Catarina, the route was through wide plains and open valleys, passing several populous towns and villages. This portion was well adapted to fast work, and 15 miles per day was averaged for several days.

In accordance with orders by wire from Mr. North, received while in Durango, the writer waited at Santa Catarina for Mr. Wilson's party, which had left Culiacan later and had taken the northern route. Mr. Wilson soon arrived, the lines were joined, and the return on horseback to Culiacan commenced. Mr. Wilson intended to examine the head waters of the Humaya River, but the total absence of trails and the fact that all of the pack and saddle animals were so feeble from the effects of cold and snow that they could scarcely carry a load on a good, well-beaten trail, caused him to abandon that project and

return by the route of his survey with the writer. Heavy rain and snow storms were encountered, and progress was slow, as it was necessary to wait for several streams to subside before they could be forded. Mr. Landreth.

The writer is of the impression that the circuit covered was 606 miles, that the latitudes and departures when calculated closed within 1 100 ft., and that the total cost of the survey was about $3 per mile.

As the survey started nearly at the sea level, crossed over mountain passes at an elevation of upwards of 11 000 ft., was through rough, broken country as well as level plains, and through tropic heat and winter's snow and ice, it furnishes an example of the adaptability of the stadia to such work.

J. B. Johnson, M. Am. Soc. C. E.—This paper is another endorsement of the stadia method of surveying, which also adds materially to the advance of the system. The fact that the stadia method was found by actual trial in easy country to be more accurate than chaining, even after the correction of several large errors caused by a miscount of the number of chains, is very significant. This has often been claimed by the advocates of this method, but probably never before proved so directly by actual trial. Mr. Johnson.

Another significant contribution to the literature and also to the practice of this method is the evidence produced of the importance of the time and method of determining the value of the wire interval, or of graduating the stadia rod. This matter has never before been clearly understood. Mr. L. S. Smith has rendered the users of the method a good service in his investigations in differential refraction, in showing the importance of choosing average conditions for standardizing the instruments.

The author's methods of work are to be commended for their accuracy, but they require greater labor of reduction than the ordinary methods. Thus it is common to read the distances directly from the rod, specially graduated for that purpose, and to read the middle wire to the "H. I." mark on the rod for vertical angle. These methods have been found satisfactory in general, and will probably continue to be used. The most important errors in the ordinary practice are a failure to graduate the rod for many readings taken under all sorts of atmospheric conditions, and reading the bottom wire too near the ground.

By far the most rapid and accurate method of reducing stadia notes for differences of elevation is by means of the Colby slide rule, which is specially constructed for this purpose. By using this the differences of elevation can be taken out as rapidly as the notes can be called off and the results written down by the assistant. This rule is equally adapted to all units or combination of units, giving elevations in feet for distances in yards or meters, or *vice versa*, as readily as though all were in the same unit of measure.

Mr. Van Ornum

J. L. Van Ornum, Assoc. M. Am. Soc. C. E.—Captain Abbot's description illustrates the adaptability of the stadia in another direction, to overcome difficulties. It is most useful in hydrographic surveys. The author has made use of the stadia on surveys of different rivers for locating the banks, and also for locating soundings varying in extent from channel depths to a complete gridiron of lines covering the whole river.*

Concerning Professor Webb's suggestion of increasing the accuracy of readings, the author would state that his own method has always been to read the intercept between the stadia hairs instead of the reading of the three cross-hairs themselves. This method reduces the readings in the field from three to one and saves the subsequent reduction to intercept. The author's experience has been that there is little need of the verification secured by reading each of the three cross-hairs, and time and effort are saved. Furthermore, in some cases reading the three cross-hairs instead of observing directly the intercept would bar the use of the stadia entirely on account of the greater deliberation necessary ; in locating soundings, for example, it is often necessary to make the distance reading in one or two seconds.

Undoubtedly the discrepancies in the barometer work given in the paper are surprisingly large, and yet with such erroneous results occurring in the hundredth case, despite the care used, it would seem that complete and entire reliance cannot be placed on barometer elevations until verified by other and accurate means.

Many engineers question the advisability of using adjustable stadia hairs, as mentioned by Messrs. Allen and Laudreth. There is certainly a chance of their changing their position, and if this should occur there would be introduced a corresponding error in all the work.

The author is inclined to think that Professor Johnson overestimates the extra labor involved in using the methods described in the paper. While wishing to be understood as not attacking the usual methods of specially graduating the stadia rods for each particular transit and of reading the vertical angle to the "H I" mark on the rod (methods which have proved very serviceable and generally accurate enough), he believes the methods given in the paper offer advantages for any survey, and in difficult work become a real necessity in eliminating errors that should hardly be allowed, viz., at a distance of 1 000 ft., with the usual stadia rod the intercept would cover so much of the rod that in changing the telescope to read the vertical angle to the "H I" point probably 3 ft. would be passed over in the movement. This, for a vertical angle of 20° would introduce an error of 2.25 ft. into the horizontal and of 1.95 ft. into the vertical component. Severe conditions may not often occur, but when encountered they may be overcome without a sacrifice of accuracy.

* See the *Journal* of the Association of Engineering Societies, Vol. XIV, No. 3.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

763.

Vol. XXXIV.—October, 1895.

THE PHYSICAL QUALITIES OF ACID OPEN-HEARTH NICKEL STEEL, AS COMPARED WITH CARBON STEEL OF SIMILAR TENSILE STRENGTH.

By H. H. CAMPBELL, M. Am. Soc. C. E.

READ AT THE ANNUAL CONVENTION, JUNE, 1895.

The strength of a bridge of a given weight and design, the weight of a bridge of a given strength and design, and the longest span possible under a given design, are all limited by the elastic strength of the material used. These well-understood axioms point naturally to the use of steel possessing a high elastic limit, but this tendency is opposed by the no less well-known fact that, as the elastic strength of steel is increased by ordinary strengthening influences, there is a reduction either in the static or the shock ductility. It is true that an increase in brittleness under shock is not always shown by the ordi-

nary system of tests, but broad generalizations from general experience have led engineers to avoid metal of high strength. Within recent years certain experiments by the United States Government upon armor plate have proved quite conclusively that an alloy of nickel with steel will give a metal possessing a high ultimate and elastic strength with an exceptional toughness. Moreover these experiments seem to show that the metal is pre-eminently fitted to withstand the effects of sudden shock.

The manufacture of nickel steel is attended by a considerably increased cost, arising from the cost of the nickel itself, which, although quoted at the lowest price that has ever been known, is now sold at 35 cents per pound. Inasmuch as the steel should contain about 3% of this element, it is easy to see that the cost, under the present conditions of nickel production, must prohibit the use of the new alloy for ordinary purposes. It has probably occurred to many engineers, however, that under certain circumstances, as in the construction of long-span bridges, the extra cost per pound might be much more than counterbalanced by the diminution in the weight, particularly of the tension members.

Before venturing to use such a new material, the conservative engineer would ask for some definite information on its physical qualities when rolled into plates, angles and rods, under the conditions of everyday practice. To furnish this information the Pennsylvania Steel Company has made a heat of nickel steel, and has rolled it into common structural shapes, the results being herewith presented. It has also made similar shapes of low phosphorus acid open-hearth steel without nickel. These two kinds of steel were rolled in the same mills, and were tested in the same machines and by the same operator, so that the results are fairly comparable. In order that such an investigation should be conclusive, it would be necessary to make a large number of heats and several tests from each heat, but that is almost out of the question in the case of the nickel steel on account of the expense involved and the fact that there is no present market for the product. On this account only 4 tons of metal were made.

Two ingots were cast of the nickel steel, one of which had a cross-section of 18 x 20 ins., and the other 16 x 20 ins. The larger ingot was rolled into four slabs with a cross-section of 16 x 5 ins., and two of these were rolled in the universal mill of the Central Iron Works,

Harrisburg, Pa., and two were rolled in the plate mill of the Paxton Rolling Mills, Harrisburg, Pa. Both mills made one ¾-in. plate and one ½-in. plate. The amount of reduction on these slabs from the ingot was not as great as might be desired, and it would have been better if a larger ingot and a thicker slab had been used; but, as the whole charge weighed only 4 tons, a larger ingot was out of the question. The smaller ingot was rolled partly into blooms and partly into billets. The blooms were rolled by the Pennsylvania Steel Company into angles of different sizes, while the billets were rolled into rivet rods at the same works.

The rods, angles and plates of the high steel containing no nickel, which is called "hard forging steel" in the tables, were rolled from such blooms and slabs as happened to be on hand at the time, and hence were from different charges. They were made from ingots varying in cross-section from 16 x 20 ins. to 24 x 32 ins.

There are also given the results obtained on a lot of steel which was used for the construction of the massive gates now being placed in the Cascade Canal locks on the Columbia River, in Oregon. There are four of these gates, each consisting of two leaves, each leaf being 51 ft. wide and from 37 to 55 ft. deep. The steel was made by the Pennsylvania Steel Company, while the gates were built by the Maryland Steel Company. The fact that the metal was of unusual strength will render the figures valuable for comparison. The specifications required that one piece of an angle should open out flat, while another piece should close shut without fracture. In the case of plates, a strip was to bend double around a curve whose radius was equal to 1½ times the thickness of the piece. As a matter of fact, each test was bent flat upon itself, and out of 3 000 tons of material there was not a failure of a test piece of angle or of plate. The specifications called for an ultimate strength between 70 000 and 80 000 lbs. per square inch, but the accompanying table gives only those heats that ran between 75 000 and 80 000 lbs. This metal is called "forging steel" in the tables.

Tables Nos. 1, 2 and 3 give the detailed records of the three grades of steel, while Table No. 4 places these records in more convenient form for comparison. Table No. 5 gives the result of applying some of the ordinary miscellaneous tests of structural steel. The elastic limit was found by the drop of the beam. This was readily determined in the case of the nickel steel and of the softer "forging steel," but was

not well marked in the plates of the "hard forging steel." In the case of the "forging steel" there are some data lacking. This arises from the fact that the figures given are records of steel made and tested more than a year ago, and there was no need at the time of having this particular information. The results are rather disappointing in showing a less superiority of the nickel steel than may be commonly supposed, Special methods of heat treatment might be made to show better results, but if this plan is followed in the case of one grade of steel, it should be followed in all others. Judging from the records given, it seems doubtful if nickel steel offers such marked improvement in quality that it can be employed profitably in ordinary engineering work.

TABLE No. 1.

Physical Properties of Acid Open-Hearth Nickel Steel.

Made by the Pennsylvania Steel Company. Composition, C, 0.24; Mn, 0.78; S, 0.027; P, 0.032; Ni, 3.25.

Shape of member.	Size of member.	Ultimate strength. Pounds per square inch.	Elastic limit. Pounds per square inch.	Elastic ratio. Percentage.	Elongation in 8 ins. Percentage.	Elongation in 2 ins. Percentage.	Reduction of area. Percentage.
Rounds	¾″ diam.	86 220	64 140	.....	20.00	34.00	45.6
	"	85 960	63 100	.....	20.00	34.00	45.5
	"	85 720	63 580	.....	20.50	34.00	48.1
	"	86 160	63 480	.....	20.25	34.00	46.1
	Average..	86 015	63 575	73.9	20.19	34.00	46.3
Angles	3″ x 3″ x ⅜″	88 600	59 720	.....	19.75	37.00	49.2
	"	85 280	57 800	.....	22.00	41.00	51.5
	3″ x 3″ x ½″	87 000	58 140	.....	23.50	41.00	50.7
	Average..	86 960	58 553	67.3	21.75	39.67	50.5
Universal plates. Longitudinal tests	⅜″ x 16″	84 080	57 320	.....	21.25	40.00	53.1
	"	85 140	59 240	.....	21.75	39.00	54.2
	"	86 480	59 000	.....	19.25	37.00	47.8
	½″ x 16″	86 200	58 260	.....	21.00	40.00	54.1
	"	86 660	58 320	.....	21.00	39.50	52.3
	"	86 080	58 320	.....	22.25	40.00	50.5
	Average..	85 773	58 410	68.1	21.08	39.25	52.0
Universal plates. Transverse tests	⅜″ x 16″	84 460	57 560	.....	14.50	27.00	40.6
	"	87 020	58 440	.....	14.50	26.00	36.8
	"	87 440	59 000	.....	15.75	31.00	39.1
	½″ x 16″	86 300	58 160	.....	18.50	34.50	41.6
	"	86 360	57 900	.....	16.25	25.00	25.6
	"	86 920	58 160	.....	19.50	30.00	33.0
	Average..	86 417	58 203	67.4	16.50	28.92	36.1
Sheared plates. Longitudinal tests	⅜″ x 44″	86 520	59 560	.....	20.50	37.00	52.6
	"	84 440	58 800	.....	19.50	37.00	48.8
	"	86 680	59 640	.....	17.00	31.50	42.8
	"	85 640	57 600	.....	16.75	32.50	40.0
	½″ x 44″	84 020	57 600	.....	20.50	39.00	52.2
	"	84 880	57 180	.....	19.75	37.00	51.5
	"	85 180	56 800	.....	19.00	34.50	50.2
	Average..	85 337	58 169	68.1	19.00	35.50	48.3
Sheared plates. Transverse tests	⅜″ x 44″	85 120	57 680	.....	17.00	26.00	34.3
	"	83 840	58 380	.....	19.00	34.00	42.5
	"	83 630	58 040	.....	18.50	35.00	42.8
	½″ x 44″	83 520	55 220	.....	16.00	34.00	48.0
	"	84 520	56 980	.....	15.50	32.00	47.0
	"	85 630	.....	.....	16.75	34.00	45.7
	Average..	84 377	57 260	67.9	17.13	32.50	43.4

TABLE No. 2.

Physical Properties of Acid Open-Hearth Hard Forging Steel.

Made by the Pennsylvania Steel Company. Composition, C, 0.30 to 0.35; Mn, 0.60 to 1.00; S, 0.03 to 0.05; P, 0.03 to 0.05.

Shape of member.	Size of member.	Heat number.	Ultimate strength, Pounds per square inch.	Elastic limit, Pounds per square inch.	Elastic ratio. Percentage.	Elongation in 8 ins. Percentage.	Elongation in 2 ins. Percentage.	Reduction of area. Percentage.
Rounds	¾″ diam.	11 114	87 900	58 630	.....	15.90	23.50	30.5
	"	11 114	87 180	57 140	....	15.75	24.00	30.3
	"	11 114	87 500	57 940	.....	18.50	27.00	32.0
	"	11 114	88 070	58 510	.....	16.63	23.25	28.3
		Average..	87 663	58 055	66.2	16.70	24.44	30.3
Angles	4″ x 4″ x ⅜″	11 114	88 000	52 640	.....	19.00	35.50	47.2
	4″ x 4″ x ½″	11 114	85 740	56 980	.....	20.50	38.00	50.9
	5″ x 3″ x ½″	11 114	89 720	52 840	.....	18.25	31.00	31.7
		Average..	87 820	54 153	61.7	19.25	34.83	43 3
Universal plates; longitudinal tests	½″ x 16″	11 233	82 900	50 440	.....	20.75	37.50	46.1
	"	11 233	83 640	50 500	.....	19.50	36.00	43.9
	"	11 233	81 780	49 550	.....	21.25	39.50	50.9
		Average..	82 773	50 163	60.6	20.50	37.67	47.0
Universal plates; transverse tests	½″ x 16″	11 233	84 940	(50 000)*	.....	19.00	23.00	27.3
	.....	11 233	84 500	(50 000)*	.....	19.00	24.00	27.7
	.....	11 233	86 080	(50 000)*	.....	18.50	22.50	27.2
		Average..	85 173	(50 000)*	(58.7)*	18.83	23.17	27.4
Sheared plates; longitudinal tests	½″	10 711	85 300	(50 000)*	.....	23.50	38.00	45.8
	.....	10 711	85 800	(50 000)*	.....	21.50	37.00	45.2
	.....	10 711	82 700	(50 000)*	.....	20.25	37.50	52.0
	.....	10 711	86 620	(50 000)*	.....	22.00	40 00	47.7
	.....	10 711	84 640	(50 000)*	.....	23.25	44.50	51.3
		Average..	85 012	(50 000)*	(58.8)*	22.10	39.40	48.4
Sheared plates; transverse tests	.....	10 711	83 700	(50 000)*	.....	21.00	34.00	35.4
	.....	10 711	84 300	(50 000)*	.....	19.50	35.50	39.6
	.....	10 711	84 380	(50 000)*	.....	24.00	36.00	35.8
	.....	10 711	83 700	(50 000)*	.....	23.75	41.00	46.9
	.....	10 711	83 920	(50 000)*	.....	22.50	38.50	41.2
	.....	10 711	85 960	(50 000)*	.....	19.50	37.00	48.7
		Average..	84 327	(50 000)*	(59.3)*	21.71	37.00	41.3

* Approximate; could not determine accurately.

TABLE No. 3.

Physical Properties of Acid Open-Hearth Forging Steel.

Made by the Pennsylvania Steel Company. Composition, C, 0.25 to 0.30; Mn, 0.60 to 0.80; S, 0.03 to 0.07; P, 0.03 to 0.06; Cu, 0.20 to 0.40.

Shape of member.	Size of member.	Heat number.	Ultimate strength. Pounds per square inch.	Elastic limit. Pounds per square inch.	Elastic ratio. Per-centage.	Elongation in 8 ins. Percentage.	Elongation in 4 ins. Percentage.	Reduction of area. Percentage.
Rounds.....	¾″ diam.	10 061	75 920	49 000	.....	24.50	.....	58.2
	"	10 020	76 750	53 430	.....	25.00	.....	58.7
	"	10 019	77 240	52 700	.....	25.00	.....	50.3
	"	10 022	77 620	51 680	.....	24.25	.....	46.1
	"	10 034	77 960	52 140	.....	23.25	.....	41.2
	"	10 068	79 120	52 360	.....	22.75	.....	51.9
	"	10 042	79 920	51 400	.....	23.75	.....	56.1
	"	10 047	80 000	51 630	.....	23.00	.....	53.6
		Average..	78 066	51 793	66.3	23.94	.....	52.0
Angles.....	3″ x 3″ x ⅜″	10 022	75 970	50 020	.....	.....	32.00	48.4
	"	10 019	76 620	50 610	.....	....	32.50	49.1
	"	10 068	77 040	48 060	.....	23.75	.....	54.9
	"	10 017	77 200	49 580	.....	.....	26.80	47.6
	"	10 020	78 020	49 450	.....	.....	31.25	47.8
		Average..	76 970	49 544	64.4	.....	30.64	49.6
Universal plates. Each figure is the average of two tests.	22″ x ½″	8 165	78 230	44 150	.....	22.7	.....	50.6
	36″ x 7/16″	10 034	78 635	51 005	.....	26.7	.....	52.0
	36″ x 7/16″	10 036	79 005	46 170	...	28.6	.....	54.5
	11″ x ⅝″	10 016	79 030	44 485	.....	26.4	.....	54.3
	36″ x ⅝″	10 056	79 460	46 590	.....	32.5	.....	50.9
	11″ x 7/16″	10 042	79 615	47 525	.....	23.8	.....	50.5
		Average..	78 996	46 654	59.1	26.78	.....	52.1
Sheared plates. Each figure is the average of two tests.	36″ x ¾″	10 061	78 100	49 400	.....	24.6	.....	55.0
	84″ x ⅝″	10 050	78 800	49 070	.....	20.5	.....	40.0
	32″ x ⅝″	10 056	78 900	50 800	.....	25.2	.....	51.4
	30″ x ⅝″	8 191	79 100	47 600	.....	22.2	.....	54.3
	32″ x ⅝″	8 187	79 210	47 100	.....	20.2	.....	49.0
	36″ x 5/16″	10 078	79 400	50 800	.....	19.5	.....	54.8
		Average..	78 918	49 128	62.3	22.03	.....	50.8

TABLE No. 4.

Summary of the Averages Given in Tables Nos. 1, 2 and 3.

Shape of member.	Kind of steel.	Ultimate strength. Pounds per square inch.	Elastic limit. Pounds per square inch.	Elastic ratio. Percentage.	Elongation in 8 ins. Percentage.	Elongation in 2 ins. Percentage.	Reduction of area. Percentage.
Rounds.....	Nickel.....	86 015	63 575	73.9	20.19	34.00	46.3
	Hard forging.	87 663	58 055	66.2	16.70	24.44	30.3
	Forging.....	78 066	51 793	66.3	23.94	.....	52.0
Angles.....	Nickel.....	86 960	58 553	67.3	21.75	39.67	50.5
	Hard forging.	87 820	54 153	61.7	19.25	34.83	43.3
	Forging.....	76 970	49 544	64.4	.....	.....	49.6
Universal plates, longitudinal..	Nickel.....	85 773	58 410	68.1	21.08	39.25	52.0
	Hard forging.	82 773	50 163	60.6	20.50	37.67	47.0
	Forging.....	78 996	46 654	59.1	26.78	.....	52.1
Universal plates, transverse....	Nickel.....	86 417	58 203	67.4	16.50	28.92	36.1
	Hard forging.	85 173	(50 000)*	(58.7)*	18.83	23.17	27.4
	Forging.....	.....	.....	.....	.....	.....	.....
Sheared plates, longitudinal..	Nickel.....	85 337	58 169	68.1	19.00	35.50	48.3
	Hard forging.	85 012	(50 000)*	(58.8)*	22.10	39.40	48.4
	Forging.....	78 918	49 128	62.3	22.03	.....	50.8
Sheared plates, transverse....	Nickel.....	84 377	57 260	67.9	17.13	32.50	43.4
	Hard forging.	84 327	(50 000)*	(59.3)*	21.71	37.00	41.3
	Forging.....	.....	.....	.....	.....	.....	.....

* Approximate ; could not determine accurately.

TABLE No. 5.

Comparative Miscellaneous Tests on the Steels in Tables Nos. 1, 2 and 3.

Shape.	Kind of steel.	Nature of tests.
Rounds	Nickel	Bent cold, flat upon itself.
	Hard forging	Bent cold, flat upon itself.
	Forging	Bent cold, flat upon itself.
Rounds	Nickel	When nicked, six bars bent an average of 57° before cracking.
	Hard forging	When nicked, five bars bent an average of 54° before cracking.
	Forging	When nicked, six bars bent an average of 35° before cracking.
Rounds	Nickel	When quenched, bent flat upon itself.
	Hard forging	When quenched, bent flat upon itself.
	Forging	When quenched, bent flat upon itself.
Angles	Nickel	Bent cold, flat upon itself.
	Hard forging	Bent cold, flat upon itself.
	Forging	Bent cold, flat upon itself.
Angles	Nickel	When quenched, bent flat upon itself.
	Hard forging	When quenched, bent flat upon itself.
	Forging	.....
Angles	Nickel	Opened out flat, without fracture.
	Hard forging	Opened out flat, without fracture.
	Forging	Opened out flat, without fracture.
Angles	Nickel	Closed about 45°, before fracture.
	Hard forging	Closed about 60°, before fracture.
	Forging	Closed shut, without fracture.
Sheared plates, longitudinal	Nickel	Bent cold, flat upon itself.
	Hard forging	Bent cold, flat upon itself.
	Forging	Bent cold, flat upon itself.
Sheared plates, longitudinal	Nickel	When quenched, broke at a very small angle of bend.
	Hard forging	When quenched, broke at a very small angle of bend.
	Forging	.....

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

764.

Vol. XXXIV.—October, 1895.

WHAT IS THE LIFE OF AN IRON RAILROAD BRIDGE?

By J. E. GREINER, M. Am. Soc. C. E.

READ AT THE ANNUAL CONVENTION, JUNE, 1895.

WITH DISCUSSION.

Since engineers have recently freely expressed their views on "Some Disputed Points of Bridge Construction," * and are priding themselves upon the strength, firmness and permanency of the structures now being built, it may be well to consider for a few moments the bridges which were built but a few years ago and stand practically side by side with the heavier constructions of to-day, being called upon to do exactly the same work. Many of these bridges were designed to carry the traffic existing at the time they were built, the actual train service having been taken as a basis for the calculation. But the railroad companies have been increasing the weight of their rolling stock step by step as occasion required, and

* See the *Transactions* of the American Society of Civil Engineers, Vol. XXVI, p. 77.

will continue this increase so long as it is more economical to operate a smaller number of heavy trains than a larger number of lighter trains, or until they arrive at the utmost capacity of their bridges. When this limit is reached further increase must cease or the structures must be strengthened or renewed, as their practical life has ended.

What, then, is the life of an iron railroad bridge? If heretofore it has been scarcely 25 years, it was not on account of the structures having failed or worn out under the loads for which they were designed, but because the loads had been increased to such an extent that the structures, generally of a flimsy character, became loose and shaky and totally unfit for the much heavier traffic at greater speed. The following table, showing the increase in weight of locomotives on the Baltimore and Ohio Railroad, indicates in a very forcible manner the

INCREASE IN WEIGHTS OF ENGINES ON BALTIMORE AND OHIO SYSTEM FROM 1835 TO 1895.

Type.	Date.	Weight in tons.
Grasshopper	1835	10.7
Winans' camel, eight wheels	1851	37.3
Perkins, 10 wheels	1863	45.4
Consolidation	1873	52.6
Consolidation	1881	54.3
Mogul	1886	56.6
Consolidation	1887	57.8
Consolidation	1888	62.5
Baldwin, 10 wheels	1890	66.5
Consolidation	1892	67.1
Consolidation	1894	80.4
Electric motor	1895	95.0

Increase in the	20	years	between	1835 and 1855	248	per cent.
" "	10	"	"	1855 and 1865	22	"
" "	10	"	"	1865 and 1875	16	"
" "	10	"	"	1875 and 1885	3	"
" "	10	"	"	1885 and 1895	43	"
Total increase in	60	"	"	1835 and 1895	651	"

Considering the electric motor which will be run over the Baltimore Belt Railroad, the increase in the 10 years between 1885 and 1895 is 75%, or in the 60 years, 783 per cent.

main reason why the lives of iron bridges, on this road at least, have been so very short—they were over-burdened. The other reasons, namely, flimsy construction and poor details, evil influences tending to a short life, are not generally met with in structures built during the past 10 or 15 years, although it must be acknowledged that some rail-

road companies which buy their bridges as so much stock in trade, without employing a competent party to look after their interests, are getting structures but little better than those used a quarter of a century ago and which develop into flimsy affairs just as soon as a little increase in traffic is felt.

Leaving such structures out of the question and considering only those which have been conscientiously designed in accordance with the generally accepted good practice, it may be said that their lives are not to be determined by years, as is the case with animals, nor by tonnage, as is the case with rails, but by the care bestowed upon their maintenance, which care presupposes an intelligent limit being placed upon the concentrated weight of engines and train loads. Having received proper attention, and the train loads having been limited to those for which the structure was designed, and barring unavoidable accidents, there is, apparently, no reason why a good iron or steel bridge should not last indefinitely. But neglect the approaches, allow low joints in the track, use rotten ties, let the metal rust, pay no attention to loose rivets, neglect the counters and laterals, keep the roller bearings clogged with dirt, in other words, place the bridges in the hands of indifferent or incompetent men, and there will be a greater certainty that they will fail before, than that they will be standing after, 25 years.

Of the bridges built during the past 15 years, the author believes he is not far wrong in stating that the greater proportion are carrying loads in excess of their specification requirements. This leads to a very important and serious question, viz., should heavier loads than those for which the bridges were designed be allowed to run regularly, and, if so, to what safe extent can these loads be increased ? If it is asserted that train loads cannot be safely increased beyond specification requirements, then experience belies the assertion. If it is acknowledged that these loads can be increased without any possible injury to the structure, it is then implied that there has been a considerable waste of material in the design, which waste cannot be considered good engineering.

It must be admitted that the usual specification stresses can be increased to a limited extent without subjecting the bridges to any immediate danger. It is being done now and has been done in the past, and the structures are still in good condition. The question for con-

sideration, therefore, is not, can working stresses be safely increased, but to what extent can they be increased ?

That there is a limit to all things, the capacity of the best bridges included, goes without saying. That there is a stress which is known to be perfectly safe as well as a stress which, if continued, will undoubtedly cause failure, is also readily admitted. There is, however, a considerable difference between the so-called safe and dangerous stresses. As it is the aim of an engineer to use his bridges so long as they are safe, but to take them down before they fall down, he must determine in his own mind the point where safety ends and danger begins, or, in other words, decide upon the safe capacity of the bridges regardless of the loads which they were designed, with micrometer refinement, to carry. Personal examinations will not always reveal overstrain in the members, as it will rot in timber and wear in rails. It is, therefore, a case of figures, experience and judgment, which appeal at times more forcibly to the conscience of the expert than to the purse-strings of the railroad company, and especially so when the figures indicate danger, while at the same time the bridge may be carrying the traffic to the satisfaction of all except the man who made the figures and who presumably knows what he is talking about.

Now, while some railroad men ignore calculations in connection with bridges under traffic, and from their practical experience, as they term it, are led to believe that a bridge which is apparently sound and has been carrying a certain heavy load has been tested or proved, and can therefore continue to carry this load until signs of failure are apparent, there are others with an equal amount of practical experience and knowledge of the experiences of others beside themselves, who know that while a bridge may be tested by heavy proof loads apparently in a satisfactory manner, it may collapse under a much lighter load without warning or giving any previous indication of failure. A young inspector may find a kinked eye-bar, a split angle, or a post indented by a collision, and honestly believe the structure in a dangerous condition, while it may be perfectly safe ; or the practical railroad man, whose experience has eased his conscience somewhat, may find the same defects in another structure, make a note to the effect that the bridge will be good for another year, and it may collapse upon the passage of the next train. A few figures judiciously considered have undoubtedly prevented unnecessary or untimely

renewal, just as surely as the indifference to what these few figures have indicated in some cases has resulted in a failure. Figures, however, must be tempered with a sound judgment. Each decision should be based upon a positive knowledge as to the stresses, details of design, material, workmanship and maintenance, combined with the necessary experience and ability to decide promptly and correctly, and should not be left to the snap judgment or exaggerated confidence in the personal experience of the county commissioner or practical railroad man, any more than to the immature judgment of the know-it-all theoretical alarmist.

Suppose that the design, material, workmanship and maintenance are of the best, that the structure is in first-class condition so far as can be ascertained by a rigid inspection, but since the bridge was built the rolling loads have been gradually increased until the time comes for the company to ask their engineer if the bridge will carry engines weighing 50% more than it was designed to carry; if not, then what is the heaviest engine that can run regularly over the bridge? These are practical questions and are being continually asked railroad engineers, who must and do answer promptly, although the answers are usually neither discussed publicly nor published, owing to a desire to avoid censure for a transgression of the good practice as to safe working stresses so carefully instilled in students' minds, so faithfully accepted when designing a new structure, and so philosophically stretched out when dealing with an old one. The whole question hinges upon what can be considered safe working stresses, which term has practically lost its former significance, owing to the different practices, since there are some specifications calling for a safe working stress of 9 000 or 10 000 lbs., and others drawn up by engineers of an equal standing give a safe working stress of 14 000 or 15 000 lbs. This large difference is, of course, due to the variable allowances made for impact, and there is practically no difference so far as the strength and weights of bridges are concerned when designed for the same engine loads. It is, however, confusing to the ordinary railroad officer who has neither the time nor inclination to look into the allowances for impact, and who cannot clearly understand why there should be such a great difference in the safe working stresses used by different designers.

Railroad bridges were formerly built with a supposed factor of safety

of five, and there are many railroad men at the present day who still believe that it would take five times the load for which the bridge was designed to break it down, which unfounded belief causes them to have an undue confidence in some very light structures. While it is well known to bridge engineers that there is no meaning whatever in the term "factor of safety," and, in fact, that there is no such a thing in connection with bridge work, there are still some who are loth to give up, and who add certain large and variable percentages to their live load stresses so as to cover the effects of impact, etc., and then assume a uniform safe working stress approaching one-half of the elastic limit of the material. It appears to the author that it is neither rational nor logical to add arbitrary percentages to the live load stresses in the hope of providing for the uncertain and erratic effects produced by a moving train. The static stresses can be approximately determined, but this percentage custom is only a crude guess which, when added to the figured static stress, makes a guess out of what was otherwise a scientific determination. Still another guess is made in assuming the safe working stress and the section of the member finally arrived at is the result of one fact, plus a guess, divided by a guess. In fact the whole method is but a conglomeration of guesses.

There are a few facts concerning material, rolling loads, and contingent effects which are known without resort to guesswork. For instance, it is known that structural material, when repeatedly stressed beyond the elastic limit, will be injured and ultimately broken. It is known that structural material when subjected to loads without shocks will not break at a stress below the elastic limit, no matter how often this load is repeated. The elastic limit of the material is known, also the approximate static stress in each member of the bridge. It is also known that the effects of impact, shocks or blows are more severely felt in the members which receive these blows suddenly and directly and which have little inertia of their own to resist or absorb them. Upon such facts and upon personal experience with bridges under traffic, not with bridges on the drawing board, should be based the foundation of the safe working stresses, which should be made to vary inversely as the known effects of shock vary, or, in other words, they should be a function of the live and dead load stresses without adding any guess to either.

This is the new method of dimensioning quite generally employed.

and the formulas based upon the foundation of facts indicate a rational means of determining sections and providing for dynamic effects, without obliterating stresses which can be calculated. Safe stresses based on the above method mean that a member subjected to a dead load only will be perfectly safe so long as the unit stress does not exceed the elastic limit, that when subjected to a live load it will be safe so long as the unit stress does not exceed one-half the elastic limit, and when subjected to both dead and live loads will be safe at a stress somewhere between that allowed for all-dead loads and all-live loads. The above remarks are admittedly ancient history, but are given here more for the purpose of having a clear understanding as to what is meant by safe working stresses than as an argument in favor of the new method.

If it is admitted that a stress just within the elastic limit is safe for a dead load and one-half this amount is safe for a live load, it must be borne in mind that such high stresses cannot be used in dimensioning bridges, because there are certain contingencies which cannot be readily recognized by any formula, but which must nevertheless receive a proper amount of attention.

First.—Structural material may have defects which cannot be discovered, no matter what care is bestowed upon the inspection and tests. Something must therefore be allowed for such possibilities.

Second.—Something is allowed for bulk, so that members will have a sensible inertia to withstand vibrations and absorb shocks, thereby avoiding flimsy construction.

Third.—It is desired to have bridges somewhat safer than just safe, as super-refinement in this direction is an absurdity.

In addition to the above, a tacit provision is made for future increase in traffic, as train loads have generally gone beyond what was expected at the time of designing. It might also be added that something is thrown in from force of habit.

No one can deny that the above three contingencies are legitimate, and that in order to provide for them a stress less than the elastic limit for dead loads and less than one-half the elastic limit for live loads must be used in considering whether or not a structure is safe or in designing a new bridge. Assuming, then, that the safe working stresses, as accepted in good practice, in addition to the above three contingencies, provide for a future increase in train loads,

and are generous as well from force of habit, it would appear that if the "force of habit" and "future increase" could be eliminated, or, in other words, if it could be determined just how much is allowed for these two items, there would not be much difficulty in fixing upon the utmost capacity of a railroad bridge of good design. Experience with structures which have been working for some years under an increased traffic without injury furnishes the only means of solving the question.

Considering the subject in relation to timber stringers, interesting and valuable data as to the unit stresses successfully allowed in the practice of the principal railroads of the Northwest have been collected and published by Onward Bates, M. Am. Soc. C. E., in his discussion on "Foundations and Floors for Buildings of the World's Columbian Exhibition," a paper read by the late A. Gottlieb, M. Am. Soc. C. E., before the Western Society of Engineers.*

The practices of ten railroads were given, in which one used a working stress between 1 700 and 1 800 lbs., one between 1 600 and 1 700 lbs., two between 1 500 and 1 600 lbs., three between 1 400 and 1 500 lbs., two between 1 200 and 1 300 lbs., one between 1 000 and 1 100 lbs. The above were white pine stringers figured in accordance with the usual practice of making no allowance for continuity over two supports, and when side stringers were used, of considering that they carried the dead loads, and that the main stringers carried the live loads. No impact was allowed. As Mr. Bates aptly says: "If experience is worth anything these figures will speak for themselves."

The author has personally examined many Georgia yellow pine trestles in which the stringers were and are still subjected to stresses of from 1 600 to 1 800 lbs., and has yet to find a single case of cracking or breaking under such stresses, although some of the stringers were eight years old and correspondingly weather beaten. There was one case in which the stresses were as high as 2 400 lbs. for a period of several years, and, while the stringers were very old, there was nothing to indicate that excessive strains had injured them. This was, of course, an isolated case, and is not given here to illustrate common practice, but rather to show what timber, sound and free from injurious knots, has stood in a long service.

Compare successful practice throughout the country with the ridiculously low safe stress of 750 lbs. for white pine and 1 000 lbs. for yellow

*See the *Journal* of the Association of Engineering Societies, Vol. X, p. 577.

pine, laid down as law by text books since the publication of Lanza's experiments, and state why these low stresses should be recommended, when experience has demonstrated that from 80 to 100% increase is perfectly safe. Do the 750 lbs. given in the books consider the future possibility of rot and the necessary decrease in section and strength occasioned thereby? If so, then it is an absurd reason, as no stress can safely be assigned to rotten timber, and it would be much better engineering to use fiber stresses of from 1 200 to 1 500 lbs., and strengthen or renew the stringers upon evidence of injurious rot, rather than to proportion for a stress of 750 lbs., in the belief that rot has been provided for. The actual practice above noted will surely admit of the very conservative conclusion that when stresses are figured from actual wheel loads and so long as the timber is sound, white pine stringers will be safe at 1 600 lbs., and that an occasional stress as high as 1 800 lbs. will do no damage. It is the author's practice to consider 1 600 lbs. for white pine and 1 800 lbs. for Georgia yellow pine as the upper limits, and to renew or strengthen as soon as the condition of the timber becomes suspicious. In designing new work it is very conservative practice to use 1 000 lbs. for white pine and 1 200 lbs. for yellow pine, and in the author's opinion 1 200 and 1 500 lbs. for white and yellow pine respectively, is a safe and economical practice, which will still admit of an increase in engine loads.

During the past few years the author has made many inspections of existing iron bridge structures, and has found cases which he considers as furnishing interesting data for consideration in connection with the utmost capacity or limit of life in railroad structures. Some of these cases are given below and the permissible unit stresses are taken in accordance with the Baltimore and Ohio specifications of 1894, which agree closely with accepted good practice. These unit stresses for wrought iron are as follows:

Tension in bars ..... $7\,500\left(1+\frac{\text{min.}}{\text{max.}}\right)$

" plates and shapes ..... $7\,000\left(1+\frac{\text{min.}}{\text{max.}}\right)$

Compression in chords, posts and main braces. $7\,000\left(1+\frac{\text{min.}}{\text{max.}}\right)-35\frac{}{r}$

Compression in girders ..... $6\,200\left(1+\frac{\text{min.}}{\text{max.}}\right)$

Shearing of rivets 6 000

Bearing of rivets 12 000

In these formulas, l represents the length in inches, and r the radius of gyration.

Case I. I *Beam Girders.*—Span, 26 ft. center to center; three 15-in., 50-lb. I beams under each rail; dead load, 350 lbs.; live load, 3 075 lbs.; total load, 3 425 lbs. per foot on one girder. Permissible unit stress in top flange, 6 830 lbs.; actual, 16 400 lbs.; overstrain, 140 per cent. Permissible unit stress in bottom flange, 8 270 lbs.; actual, 16 400 lbs.; overstrain, 98 per cent. The bridge was in good condition.

Case II. *Deck Plate Girder.*—Span, 20 ft. center to center; one web, 15 x $\frac{1}{2}$ ins., and four angles, 5 x $3\frac{1}{2}$ x $\frac{5}{8}$ ins.; rivets, $\frac{7}{8}$ in. diameter, with pitch from 4 to 6 ins.; dead load, 300 lbs.; live load, 3 420 lbs.; total, 3 720 lbs. per foot of one girder. Upper flange; safe stress, 6 730 lbs.; actual, 17 280 lbs.; overstrain, 156 per cent. Lower flange; safe, 8 150 lbs.; actual, 19 660 lbs.; overstrain, 141 per cent. Rivet bearing; safe, 5 250 lbs.; actual, 10 540 lbs.; overstrain, 100 per cent. The above girder was figured on the supposition that bending strains were resisted entirely by the flange angles. The bridge was in good condition.

Case III. *Howe Truss.*—Span, 84 ft.; eight panels, depth 16 ft. 3 ins.; first panel rods, two rods 2 ins. diameter, not upset; dead load on rods, 20 500 lbs.; live load, 64 200 lbs.; total, 84 700 lbs. Safe stress, 9 300 lbs.; actual, 17 600 lbs.; overstrain, 89 per cent.

Case IV. *Bollman Bridge.*—Stress on main rods, considering that panel rods do not relieve them; safe, 9 000 lbs.; actual, 22 000 lbs.; overstrain, 144 per cent. Stress on panel rods, considering that they do relieve the stresses in the main rods; safe, 9 000 lbs.; actual, 16 500 lbs.; overstrain, 83 per cent. The over-strained members in this bridge were in good condition, but as it is very uncertain to what extent the panel rods relieve the main rods, the bridge was condemned.

Case V. *Warren Truss of 155-Ft. Span.*—Main ties; safe stress, 10 000 lbs.; actual, 19 200 lbs.; overstrain, 92 per cent. Second brace; safe stress, 6 080 lbs.; actual, 9 690 lbs.; overstrain, 59 per cent. Third brace; safe stress, 2 100 lbs.; actual, 9 320 lbs.; overstrain, 344 per cent. The cross-ties of this bridge rested directly on the bottom chord, producing a transverse stress in addition to the direct stress, some of which stresses were as follows: Safe, 8 000 lbs.; actual, 19 340 lbs.; overstrain, 142 per cent. This bridge was built in 1871. The third braces were 6-in. channels $25\frac{1}{2}$ ft. long, and had a value of $\frac{l}{r} = 140$,

which would not be permitted at the present day in any first-class structure. The structure was condemned, its life being about 20 years.

The foregoing five cases of overstrained bridges cover pretty well the ordinary conditions as to style, and were selected especially for the purpose of embracing the field of beam girders, plate girders and truss bridges. They are not isolated cases, and many more illustrations of the same kind could be brought forth if necessary. Before drawing any conclusions, however, it may be well to mention that the author has seen the vertical end posts of a deck Whipple truss, composed of two 8-in. channels weighing 12½ lbs., latticed by 2 x ¼-in. bars, flat at one end and hinged bearing at the other, with a total length of 33 ft. 4 ins., and a value of $\frac{l}{r} = 132$, bend sideways 1½ ins. when overstrained 52%, when figured according to the Baltimore and Ohio formula for compression members, or overstrained but about 8% when figured by Rankin's formula for columns flat at one end and hinged at the other. The posts were connected in pairs and both bent in the same direction. They carried the load on one panel only.

The author has also known short floor beam hangers which were bent around a pin in the form of a yoke, to snap square across through both arms at a strain of 12 930 lbs. per square inch, or at an overstrain of 80 per cent. This, however, was an isolated case and was undoubtedly due to a very poor quality of iron, as the breaks occurred in the body where there were 6 sq. ins., while there were less than 5 sq. ins. through the thread.

Main rods in Bollman trusses have snapped short off without reduction of area or elongation, under a stress of from 25 000 to 27 000 lbs., the panel rods not considered as carrying any of the load, or at an overstrain of from 138 to 157 per cent. As it is almost certain, however, that the panel rods did carry a part of the load, this overstrain would be considerably reduced. Again, while the author has never yet met with an I-beam girder which failed in service, either by buckling of the top flange or tearing apart of the lower flange, he has seen many cases where the webs have split open for a distance of 3 or 4 ft. when the train loads were but 50% in excess of what the beams were intended to carry.

No conclusions could be drawn from the particular cases cited had they been isolated, but as stated before they are not isolated and were

selected for illustrations of numerous similar conditions found in practice.

One thing is pretty clearly brought out, and this is that if the limit of elasticity of wrought iron is considered as 26 000 lbs., and an all-moving load is held to produce twice the strain that is caused by an all-dead load, then something is radically wrong in the usual method of figuring stresses from concentrated wheel loads; otherwise, each of the structures mentioned should have shown a permanent set, which they did not. If the wheel method of figuring stresses is correct, then the preceding examples demonstrate without question that an all-moving load does not produce twice the stress caused by an all-dead load. It is positively certain that the method of figuring stresses from concentrated wheel loads causes strains so far above what really exist as to almost class this method as unscientific, because there can be no concentrated loads when there are stiff rails and cross-ties spaced at close intervals placed between the load and the supporting girder. A distributed load equivalent to the sum of the concentrated wheel loads of the engine would give stresses nearer the truth. While it has been, and is generally, considered that a suddenly applied load produces about twice the stress of the same load at rest, and that an engine moving over a bridge is a suddenly applied load, there is considerable doubt in the minds of practical bridge men as to the truth of this assumption. In fact bridge men who are continually looking after bridges in service are strongly inclined to believe that, so far as strains are concerned, the mere fact of a train rushing across a small opening does not in itself cause any greater strains than should the same engine cross slowly, as it takes time for strains to be felt and transferred through the body of the supporting girders. A rushing engine gets across in so short a time as to probably eliminate all impact. Low joints in the track and unbalanced drivers do, beyond all question, cause blows to be delivered to the supporting girders, and the greater the speed, the more force there is in these blows, and considered in connection with unbalanced drivers, low joints and rotten ties, great speed does cause impact. But with the track in proper condition and fairly well-balanced drivers, the author believes that the quicker a train gets across a weak bridge of very small span, the better it will be for the bridge. This of course does not apply to truss bridges, long trestles or viaducts.

As bridge engineers in practice who are frequently called upon to answer the question suggested in the subject of this discussion must base their judgment largely upon what has been done and should be somewhat consistent in their opinions, it may be well to arrive at some understanding as to what can be considered an upper limit in stresses produced by the constantly increasing engine loads. When these limits are reached it is time to strengthen or renew, and it must be distinctly understood that the suggestions advanced by the author apply only when stresses have been determined from actual wheel loads, when the structures are of good design and in good condition, and where details have been made to develop the full strength of the members. Details of connections have heretofore been the weakest parts of bridges, are the first to cause trouble, and, as a rule, must be strengthened long before the main sections are in any danger.

While no engineer would for an instant think of using the stresses suggested below in making new designs, it is necessary at times to use them in old work, and while it is believed that these stresses will be safe for a long period, there is no certainty as to how long. Whenever permitted, the structure should be under very careful supervision. There is no doubt that modern steel bridges are practically permanent so long as they are properly maintained and so long as the loads do not exceed the specified requirements, and if there was any assurance that they would be equally permanent under an increased loading of 50 or 75%, the fact would soon be recognized by increasing the present accepted unit stresses and adopting lighter structures.

The following unit stresses are suggested as the highest permissible for a constant service. An occasional load a little in excess of these limits will do no harm.

a. Timber beams free from loose or rotten knots and in sound condition. Unit stress on extreme fibers; white pine, 1 600 lbs.; long leaf yellow pine, 1 800 lbs.

b. Rolled iron beams, braced firmly together, in groups of two or three, so as to form a compound girder, the unsupported lengths of compression flange not exceeding 16 times its width. Unit stress,

$$13\,000 \left(1 + \frac{\text{min.}}{\text{max.}}\right).$$

As the webs are likely to split open at a less stress than indicated above, beam girders should be closely watched whenever stresses exceed

the specification requirements by 40 per cent. These cracks, however, can be readily prevented by means of stiffening plates extending from flange to flange and riveted, both to the flanges and to the web at each end of the beam. Cracked beams will still do good service when the cracks are strongly spliced.

c. Plate girders with an efficient system of lateral bracing, stiffened web, rivets correctly proportioned, cross-ties resting on the upper flange or on an iron floor system. Main girders, compression flange, $11\,000\left(1+\frac{\text{min.}}{\text{max.}}\right)$; main girders, tension flange, $13\,000\times\left(1+\frac{\text{min.}}{\text{max.}}\right)$.

All connections of stringers to floor beams and floor beams to girders in through bridges should be strengthened when the shear exceeds 9 000 lbs. on the diameter of rivets, or the bearing exceeds 18 000 lbs., and all loose rivets should be replaced by tight ones as soon as discovered. When cross-ties rest directly on the lower flanges, an increase in flange stress of 50% would be uncertain and liable to cause trouble.

d. Truss bridges.—In Bollman bridges the main rods will be safe at a stress of 20 000 lbs. when the panel rods are in good adjustment and are capable of aiding the main rods. When the panel rods are loose or cannot be adjusted, owing to rusty threads or other causes, the stress should not exceed 16 000 lbs. per square inch.

In Pratt or Warren bridges, the stresses in the main ties, lower chords, long suspenders or counters should not exceed $13\,000\times\left(1+\frac{\text{min.}}{\text{max.}}\right)$. Stresses in short bar hangers should not exceed 12 000 lbs. Upper chords and posts, when the ratio of the length to the least radius of gyration does not exceed 115 or 120, can have the stresses increased to $12\,250\left(1+\frac{\text{min.}}{\text{max.}}\right)-61\frac{l}{r}$.

If the above conclusions are accepted, then the life of an iron bridge of moderate span will have reached its limit when train loads have been increased between 80% and 90% beyond what the bridge was designed to carry. They must not be construed to mean more than they affirm, and when used in actual practice it must be remembered that there are other parts of a bridge besides the ties and columns, and that while these main members may be reasonably safe, the structure, depending, as it does, upon other features as well as its main members, may be treacherous.

DISCUSSION.

Mr. Morison. GEORGE S. MORISON, President Am. Soc. C. E.—The conditions which the author has brought out, namely, that a bridge will do good service under strains which are very much greater than it was intended to bear, show what is now one of the greatest defects in bridge specifications. It will not do to say that no bridge has ever failed because the metal in a main member was overstrained, but this is an extremely rare occurrence; the difficulties, wear and everything else, occur in the details. Bridges are generally designed for a certain strength and proportioned so that their metal shall not be overstrained, but there is scarcely a specification drawn which limits the distortion which a bridge may be permitted to undergo. Yet it is perfectly possible to design a bridge in which the metal of every member would be strained up to 20 000 lbs. to the square inch, and have a bridge which would wear better, last longer, and be a better structure than another one in which the strains are limited to half this amount, even though the second structure be proportioned properly in all its details. A difficulty in many bridges is that the trusses, as well as the different members, can change their shape too much without passing proper limits of strength, and this is a point which has scarcely ever been provided for in specifications. An examination of an elevated railroad structure, which was actually erected and in which everything was properly proportioned so far as strain was concerned, showed that the structure, without overstraining any metal, could move 9 ins. out of line horizontally; nobody would think this a proper condition to allow. This is a point which has not been given proper attention, although there are many things about specifications which go into minutiæ.

The matter of counter-balancing has been referred to. Many locomotives, which theoretically have 15 000 lbs. on an axle, really when moving at high speed exert pressures varying from 32 000 to minus 2 000 lbs., which is rather an unfortunate condition; but still these engines are running all the time. If a structure is so designed that it keeps its shape under the passage of a train, or if it changes its shape in so gradual a way that the change is not felt, it may be all right even with very high strains.

In an important bridge an examination was made and the pins were found to have a theoretical bending strain in extreme fibers of from 175 000 to 250 000 lbs., and yet when some of those pins were taken out they were perfectly straight. Unfortunately no observations were taken to show the actual variations of strain in the bars attaching to the pins, but if the bars had been examined under strain with a micrometer and the strains in them had been found to differ not more than 15% from a mean, the bridge should have been accepted as a safe

structure, even though the theoretical bending strains in the pins exceeded 300 000 lbs. to the square inch. It was simply a case where the theoretical strains in the pins are of no real importance, provided the pins stay straight; in other words, they do not exist if the pins are straight. Mr. Morison.

A great many of the older bridges were designed with very high strains. Albert Fink, Past-President Am. Soc. C. E., 25 years ago considered 14 000 lbs. to the inch a perfectly proper strain to put on the tension members of his bottom chords, and none of his bridges proportioned in this way failed to give good results, though most of them have now been superseded by structures adapted to very much heavier rolling stock.

T. C. Clarke, M. Am. Soc. C. E.—Among the most remarkable instances in the United States of bridges built to carry light loads, which now support trains much heavier than those for which they were designed, are the riveted lattice bridges constructed on the New York Central Railroad about 1869 or 1870. For 25 years they have stood with but little reinforcement. There have been a number of collisions on them, but in no instance, so far as the speaker knew, had one of them yielded or broken. This is a very remarkable record, in view of the great increase of rolling loads. The speaker had watched the passage of trains over some of these bridges and attributed their great strength and durability to the fact that the strains are diffused instead of being concentrated. The practice of the designers of these bridges was to place the diagonals very close together as compared with the modern long panels. This led to diffusing the strains over a large number of points rather than concentrating them on a few, and this is one of the causes which make the bridges stand so well. This form of design, followed to its logical conclusion, leads to plate girders where a continuous web is attached to the chords by rivets very close together. Experience shows that the durability of plate girders is greater than that of any other form of construction. Mr. Clarke.

Joseph M. Wilson, M. Am. Soc. C. E.—The speaker did not believe that iron bridges give way materially from deterioration or wearing out, but considered that the troubles are occasioned by overloading or by defective design or construction. There were bridges constructed on the Pittsburg Division of the Pennsylvania Railroad, when it was first built, which were loaded above the original estimates at least 50% for 15 to 20 years, but they withstood the extra service without any difficulty. They were subsequently doubled up and used for a number of years, and finally stone bridges were put in their place. Material taken from these old bridges after 20 years of service was found on testing to have suffered no deterioration of any nature. During the civil war there was frequently occasion to estimate the increased loads which the Pennsylvania system of bridges would sustain safely, Mr. Wilson.

Mr. Wilson. and the speaker often had no hesitation in allowing an overloading of 50 per cent. There were many plate girder bridges of various spans, with depths of 5 to 5½ ft. They were built of very thin material, the webs being only $\frac{3}{16}$ in. thick, and the verticals of cast-iron, held by three bolts, acted merely as separators between the upper and lower flanges. Although these bridges rendered good service for many years, they could not have withstood rigid scrutiny under the present rules for calculating such structures.

CORRESPONDENCE.

Mr. Stanwood. JAMES H. STANWOOD, M. Am. Soc. C. E.—Near the end of the paper the following statement appears: "There is no doubt that modern steel bridges are practically permanent so long as they are properly maintained * * * " It is the object of the following notes to emphasize by mentioning some actual cases of neglect the importance of protecting iron and steel structures against rust.

Many articles have been written in recent years pointing out in no uncertain manner the grave dangers to which metallic structures are subjected by corrosion, yet instances are not lacking where authors have expressed the opinion that these dangers are greatly overestimated and that they do not call for serious consideration.

The well-known experiments of Mr. Mallet, of England, made as early as 1840, show that good iron will lose in thickness in 100 years, 0.215 in. in clear sea water, 0.404 in. in foul sea water, and 0.035 ins. in clear fresh water.

While it is held by some that sea water is a very active corrosive agent, others have observed that iron rusts no faster in clear sea water than in the air of a smoky city. Foul water is one of the most active agents of corrosion, as the writer has observed in the case of certain beams, etc., used in connection with drainage works. Drippings of brine from refrigerator cars have caused much trouble, rusting rapidly parts of the bridges over which they pass.

Smoke from locomotives, causing serious rusting of overhead bridges, roofs of round-houses, train-sheds, etc., is one of the most important elements against which the bridge engineer has to contend. Just what is the cause of the corrosion is not fully agreed upon. Many (including the writer, who has found sulphate by chemical tests in rust caused by smoke fumes) believe that the sulphur in the coal is changed by combustion to sulphurous acid, which is transformed in the presence of exhaust steam to sulphuric acid, which, though weak, certainly does have a very destructive effect, especially if confined more or less and allowed to remain in contact with iron or steel.

Others consider that the immense volume of carbonic acid gas emitted by the locomotives is the active agent of corrosion. Whatever the cause, the fact remains that locomotive fumes are a serious menace to the life of iron or steel structures. Mr. Stanwood.

The following extreme instances have recently come under the writer's notice and may be of interest:

In 1867 the late E. S. Philbrick, M. Am. Soc. C. E., constructed a plate girder highway bridge in Boston, Mass., carrying a street over an important railroad. The bridge crossed a part of the yard including certain tracks on which a large amount of shifting was done, consequently the under part of the bridge was seldom free from smoke fumes, since the floor was so made and the location was such that there was little chance for the smoke to clear. The head-room was a little more than 14 ft. When the bridge was erected it was painted with zinc paint covered with two coats of mineral paint. So far as is known this bridge was not painted for 20 years, when it was thoroughly examined and cleaned, and a coat of red lead applied to the corroded portion. At that time the bridge was so badly rusted in spots that its safety was considered doubtful. In 1889 it was removed and the writer had an opportunity to obtain several pieces, illustrations of which are presented in Plate V.

Fig. 1 represents a portion of the lower flange of a main girder at the abutment. Fig. 2 is an end view of the same specimen. The angle irons were originally $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{3}{8}$ ins., with an area of about 2.48 sq. ins. each. The area of the most corroded angle is 1.33 sq. ins., showing a maximum loss in 22 years of 46.4%, an average of 2.1% per year. The area of both angles at the same section after corrosion was 3 sq. ins., a loss in 22 years of 39.5%, or an average of 1.8% per year. Fig. 3 represents a section of the web of the main girder, showing a perforation by rusting. This web was originally $\frac{7}{32}$ in. thick. Fig. 4 shows a batten which was exposed to the fumes all these years; originally as thick as the web, it is now nearly perforated. The protection afforded by the rivet heads can be clearly seen.

These examples may be considered as extreme cases of rusting, for probably no bridge is ever likely to be more abused than this one was by neglect.

A few hundred feet from this bridge was another crossing the same railroad. It was a Boles patent bridge of a complex construction, built in 1864–65 and removed in 1890. Fig. 5 represents a portion of a cable of seven strands, each strand composed of seven wires of No. 6 Am. W. G. This cable, which was not in the original patent, was provided with screw adjustments and was doubtless added to strengthen the lower chord. It was entirely eaten away as shown, at a point over one track, in certainly less than 25 years. Fig. 6 shows the end connection of a truss rod for a wooden floor beam. The

Mr. Stanwood. upright piece in the view was originally $1\frac{3}{4} \times \frac{5}{16}$ ins., or 0.55 sq. in. in area, and now its area is 0.26 sq. in., or a loss of 52.7% in 25 years, an average of 2.1% per year. Fig. 7 represents two hanger bolts originally 1 in. in diameter. The present area at the smallest diameter is 0.37 sq. in., a loss of 52.2% in 25 years, an average of 2.1% per year. Fig. 8 is a view of still another bridge in the same neighborhood, existing under similar conditions. The large hole in the web can be seen.

The above illustrations may be a better argument than words in favor of careful and frequent inspection, and the application of the best preservative materials to exposed iron and steel in engineering structures. It will not be disputed that a large part of the lives of the bridges mentioned in connection with the illustrations was wasted by neglect.

Mr. Bates. ONWARD BATES, M. Am. Soc. C. E.—The life of an iron bridge will be definitely known when some of them have worn out under the conditions of usage for which they were designed. The writer does not know of any iron bridges which have failed under such usage, except such as would be thrown out of this discussion on account of faulty design or workmanship. The author's statement that iron bridges properly cared for and not overloaded should last indefinitely is warranted by the lack of contrary evidence, and since a limit to the life of many iron bridges has been found, he naturally looks to the cause of the limitation, and finding it to consist in excessive loading, he draws the logical conclusion that the life of a bridge is to be measured by the amount of increase in loads. Engineers who have charge of the maintenance of railway bridges can appreciate that conclusion, for the increase in weight of rolling stock constantly brings up questions of the strength of existing bridges.

These questions will recur so long as rolling stock is designed with no other consideration than to obtain the maximum pulling power of engines and carrying capacity of cars. In determining the probable ultimate strength of an iron bridge, the author's conclusion that loads may be increased to 80% or 90% beyond what the bridge was designed to carry is well enough for the net sections of the principal members; and he properly adds, in the same sentence, and by way of conclusion, "that while these main members may be reasonably safe, the structure, depending as it does upon other features as well as its main members, may be treacherous." It is these "other features" which cause the engineer of maintenance to have uneasy dreams, and as each of them is a problem which requires separate treatment, no general formula can be given for the life of an iron bridge. The problem that engineers are most interested in is to obtain the longest service from bridges, and the writer ventures the following remarks, giving some of the causes which tend to shorten the life of a bridge, and some of the means which may be taken to prolong it.

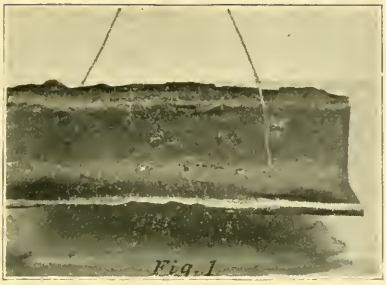


Fig. 1

Fig. 2

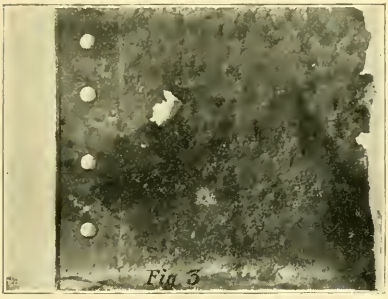


Fig. 3

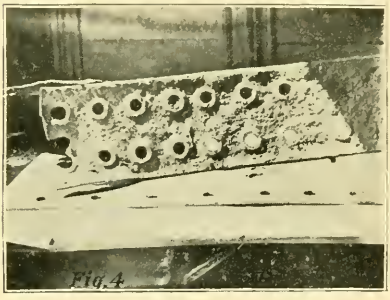


Fig. 4

Fig. 5

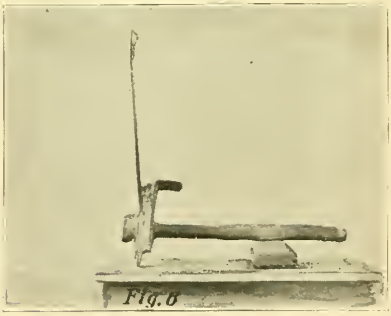


Fig. 6

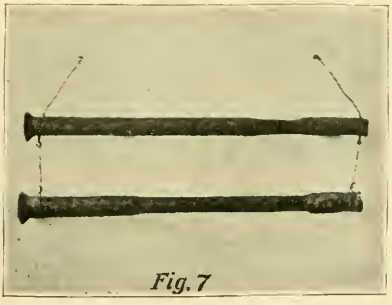


Fig. 7

Fig. 8

Mr. Bates.

The excessive loads to be provided for may be classed as regular and unusual. Regular loads are heavy locomotives and loaded cars in the regular service over the line upon which the bridge is located Unusual loads are experimental locomotives, locomotives in transit as freight, rotary snow plows, unusually heavy loads of boilers, machinery, stone, etc., and bad loading of cars, throwing excessive loads upon the trucks. It is not the weight of a train which puts the maximum stress upon the main members of a bridge that is to be feared, but rather the local loads which bring the maximum stresses upon the "other features." What is probably one of the worst cases of local loading is a locomotive hauled "dead," with driving wheels uncoupled and hammering the track at every revolution. The "other features" in bridges which are to be watched can scarcely be enumerated, for they are to be summed up in the phrase "details and connections."

The statement that the best design is the one which gives the greatest strength and durability will apply with full force to bridges which are to be built, but only to a limited extent to existing bridges. With bridge iron at $40 per ton, there is no excuse for building bridges in which there will be loose rivets or other weak spots, under even excessive service. Unfortunately the proportions of existing bridges stand in the way of adding material *ad libitum*, but in many cases it is possible to strengthen the weak portions of an existing bridge and greatly increase its life. How to do this can only be determined by a study of the bridge in question. A bridge may be strengthened while in service by reinforcing its members, by adding members, and by cushioning those members which are subject to sudden loading. To illustrate the latter method of strengthening, the writer may instance the application of heavy fir stringers of two panel lengths and breaking joints over the floor-beams of some light truss bridges. These timber floors absorb impact effects, as well as distribute the rolling load, and the comparative smoothness of riding over them is very noticeable. The cushioning effect of timber floors and wall plates is very beneficial, and these are often worth more than the cost simply as cushions.

Since the author has drawn some deductions from a statement by the writer, giving working stresses in pine stringers, the latter thinks proper to take exception to the former's statement, that in fixing the section of pine stringers it is absurd to consider and provide for the future possibility of decay. On the contrary, he deems it more reasonable to provide for the decay which is sure to occur than to put an excess of section in an iron bridge to provide for a future load which is only a probability. Timber stringers cannot be renewed with economy as soon as they begin to deteriorate. The author's figures, "unit stress on extreme fibers; white pine, 1 600 lbs.; long leaf yellow pine, 1 800 lbs.," are safe for "timber beams free from loose or rotten knots and in sound condition." But it must be remembered

Mr. Bates. that timber is not made by a mechanical or chemical process, that it is a natural product and must be taken as it grows, and in proportioning bridge stringers it is well to consider the primary defects of the material, such as knots, shakes and sap, and the defects occurring in service, such as season checks and decay, and provide an excess of material to compensate for deficiencies.

Mr. Purdon. C. D. PURDON, M. Am. Soc. C. E.—The title of the paper might read, "How much will iron bridges carry safely over and above the loads they are supposed to be designed for?" as this is what the author investigates, and his conclusion would seem to be that when the loads have reached above what might be carried safely permanently, it is time to renew the bridge. The object of the paper is thus to ascertain what strain may be safely carried by the material used in the bridge, and also, what strains a certain weight of engine will produce. The latter part seems to be the more important, as it is recognized that an engine moving over a bridge will produce strains much in excess of what would be produced by the same engine at rest.

The author says that there is considerable doubt in the minds of practical bridge men as to the assumption that a suddenly applied load produces about twice the stress that the same load would produce at rest, but some good authorities are satisfied that it is so, nevertheless. A paper by George S. Morison, M. Am. Soc. C. E., which the writer has not at hand, but quotes from memory, concludes that the manner of balancing the driver of an engine can, at high speeds, cause twice the stress it would at low speeds. C. C. Schneider, M. Am. Soc. C. E., provides in a late specification a graduated table of impact to be added, running almost as high as 100% on short spans, and S. W. Robinson, M. Am. Soc. C. E., in his report to the Ohio Railroad Commissioners, 1884–85, comes to the conclusion that all spans should have not less than 50% added for impact, unless the truss were so designed as to avoid vibration.

The writer knows of some floor beams of white pine which have carried 1 600 lbs. stress for years and which are still in good condition, and 1 250 lbs. is often used for trestle stringers, but J. B. Johnson, M. Am. Soc. C. E., in his tests for the United States Government found a variation of nearly 100% in the sticks cut from the same tree.

The load proposed for plate bridges is $13\,000\left(1 + \frac{\text{min.}}{\text{max.}}\right)$, and taking the example he gives of an old girder (page 303) this expression would give $13\,000\,(1 + \frac{3\,000}{37\,250}) = 14\,040$ lbs. for the allowable strain. This bridge was already carrying 19 650 lbs. and was in good condition. Professor Robinson in his report above quoted considers that 15 000 lbs. could be used. If all of the strains could be accounted for, the unit strain could approach very much closer the elastic limit of the material than now allowed.

The writer has lately investigated the effect of a rolling load 50% above that specified. In the end post, to increase the load 50% and allow nothing for impact would cause a stress of 8 600 lbs. per square inch, while to make the same increase and add 100% to the increased load for impact would cause a strain of 15 460 lbs. per square inch on a column which has an ultimate strength of 35 486 lbs., according to Osborne's tables. On tension members of the web, adding 50% to the moving load and neglecting impact gives 13 715 lbs. per square inch, and adding 100% to this increased load for impact gives 24 930 lbs. per square inch. On the bottom chord, adding 50% as before, gives 13 945 lbs. per square inch., and adding 100% to this gives 25 000 lbs. per square inch. For the bottom flange of floor beams, adding 50% gives 15 000 lbs. per square inch, and adding 100% for impact gives 27 900 lbs. per square inch. The material is soft and medium steel with an elastic limit of 30 000 and 35 000 lbs. Mr. Purdon.

An examination of the New York Railroad Commissioners' report on bridge strains, published in 1891, shows that Howe truss rods carried 14 000 lbs., lattice girders had angles under a tension of 10 300 lbs., and trusses had tension members of the web at 13 000 lbs., and counters as high as 15 000 lbs. per square inch. In one or two cases, counters ran as high as 30 000 lbs. per square inch, but these must have been slack.

The author confines himself to bridges lately built, but it seems to the writer that the question, "Will the bridge carry engines 50% heavier than it was designed for?" is more likely to be asked about old than new bridges, at least of the present generation of bridge engineers. It has been the writer's experience that in old bridges when calculated for present loads, the trusses show up fairly well excepting the hip vertical, and the weakest parts are the floor and its connections, so that each bridge would have to be examined for its weakest point before the question could be answered, and as the weakest point governs the bridge, so does the weakest bridge govern the division. Almost any road, except a very new one, is likely to have bridges for many different loads, and the question of what engines can be used will depend more on the old bridges than on the new ones. Figures alone will not be sufficient, for the physical condition of the bridge should be ascertained in addition to knowing what the shear and bearing per rivet is in a connection. The condition of the rivets should also be known.

G. B. Hazlehurst, M. Am. Soc. C. E.—The paper brings up a subject which caused the writer many hours of anxious thought when in charge of a large number of old bridges, most of them seriously overstrained. The question as to what limit of strain should be allowed in different members before condemning the structure is one that has received a great deal of attention on his part. In connection with the late James L. Randolph, M. Am. Soc. C. E., it was finally de- Mr. Hazlehurst

Mr. Hazlehurst

termined that the admissible increase in strain in a bridge that was otherwise in good condition might be placed at 50% above the unit strain usually considered as allowable with the ordinary factor of safety of five; or to place it more plainly, assuming the safe strain to be 10 000 lbs. per square inch for tension members, the limit was fixed at 15 000 lbs. per square inch, up to which point a member was allowed to do its daily duty without either strengthening or condemning the structure as too old. For compression members, the allowable strain per square inch usually given by the ordinary compression formulas, with an increase of 50%, was the limit fixed upon. After these strains were reached, a structure was usually trestled or strengthened temporarily if possible, awaiting its turn for replacement. Under this system, the writer carried many old bridges for several years without a single accident of any kind and with entire safety, proving to his own mind that the limit fixed was well within the limit of safety and of fairness to his company.

A nice point is before an engineer when he decides such a question; he does not wish to run any risks as regards destruction of property or loss of life, and at the same time it is important that his company should be put to no unnecessary expense. Observation of a structure under load usually influences an engineer in diagnosing the case before him, quite as much as the general appearance of a patient influences the treatment prescribed by a doctor. The writer has seen bridges show elasticity and stiffness under loads straining the tension members as high as 18 000 to 20 000 lbs. per square inch and the compression members in about the same proportion, while other bridges of inferior design would not show as well with a stress 50% less; consequently, observation is quite important in addition to an analytical determination of the stress in a structure. The writer is of the opinion that engineers to-day become a little timid as soon as the usual factor of safety is exceeded. He has known bridges to carry a heavy traffic for many years with entire safety, with a unit stress that would horrify a man not familiar with, or one who was not accustomed to watch, the behavior of structures under heavy loads.

With reference to the point brought up in the paper in regard to deflection of bridges under static loading and under rapidly moving loads, the experience of the writer agrees with the author's views. Some years ago, the late James L. Randolph, M. Am. Soc. C. E., experimented with a span of about 100 ft. length, placing on the structure an engine and train load, and observing the deflection while standing. The train was then run across the bridge at varying speeds, and at a speed of about 30 miles an hour the deflection noted was only about one half of that shown when standing. The writer has not the exact data to refer to in the case, but the results were substantially as stated, and the probable reason for them was that it takes time for the

strains to be diffused through a structure; a train rapidly rushing across a bridge does not really give time for all the members to be sufficiently affected simultaneously to produce a maximum result. Mr. Hazlehurst

M. E. Yeatman, M. Am. Soc. C. E.—Some remarks giving for comparison the experience and practice of the railroad with which the writer is connected may be of interest as bearing out in the main the conclusions in the paper. The weight of the locomotives of the Norfolk and Western Railroad Company has increased during the last 20 years in a very similar ratio to that on the Baltimore and Ohio system. This increase, and not any actual defect or wearing out, has limited the life of the iron bridges. Mr. Yeatman.

The oldest bridge still in service as originally constructed was built in 1878. The strains in this bridge are about up to the assumed limits of safety as stated below, but the structure, which is a Warren truss with all the members, tension as well as compression, of rigid form, is remarkably stiff and free from vibration, and shows no signs of weakness. Another bridge of the same date, a deck Warren truss, has had the top chords reinforced on account of the strains from bending and compression combined passing the limit laid down, otherwise the trusses are in service as built.

Bridges built in 1880 were designed for consolidation engines with 20 000 lbs. on each axle, and are still in service, though not allowing much margin for any further increase of rolling loads.

Bridges built in 1884 were designed for loads of 24 000 lbs. per axle; those built in 1889, for 30 000 lbs. per axle; those in 1891 and since, for 34 000 lbs. per axle, or 40 000 lbs. on each of two axles, this last specification giving a margin of from 6% to 10% over the heaviest engines in actual use.

The iron bridges of earlier date than 1878 in service on this system have been strengthened in all cases, or doubled up by putting three or four trusses in place of two to a track, or else are supported by trestles at two or more panel points. The general rule adopted for a guide in this matter has been to strengthen or replace bridges whenever the strains on truss members, under the heaviest engines, exceed by more than 50% those allowed by specifications; plate girders, when they exceed 15 000 lbs. per square inch on net section in tension; and stringers (yellow pine), when they exceed 2 000 lbs. per square inch. Of course room is allowed for judgment. Individual cases and several plate girder bridges showing less than the above strains have been trestled on account of visible permanent deflection, due, probably, to defective work in the riveting.

On pins, as calculated for bending, and on the bearing area of rivets, somewhat more latitude has been allowed, as is justified by the observed facts on a bridge in Pennsylvania (since replaced), which carried for years without visible ill effect loads producing a calculated

Mr. Yeatman. pressure on the bearing area of rivets in webs of floor beams ¼ in. thick of 36 000 to 40 000 lbs. per square inch, and a strain equal to 82 000 lbs. per square inch when calculated in the regular way. It is hardly to be supposed that this strain was actually borne by the pins, but it could only have been avoided by a very unequal distribution of strain between the chord eye-bars, or over the surface of the eye-bar pin holes.

Mr. Le Conte. L. J. Le Conte, M. Am. Soc. C. E.—The life of a railroad bridge depends upon how rapidly the future weights of engines and train loads are increased over and above those assumed at the time of its construction. This is a problem which is becoming more and more important each year. It is a serious question for the engineer to face, for the reason that this increase in the live load is a matter which is largely beyond his control. The author calls attention to the surplus iron in existing bridges, as proved by the great increase in engine and train loads without serious results, and comes to the conclusion that there has been a waste of material in the original design. The inference to be drawn is that there is too wide a margin between safe stresses and dangerous stresses. In the case of heavy bridges such as the Forth Bridge, the dead load is the only factor used in determining the size of members, the live load effect being a nonentity. Here the engineer can estimate with considerable certainty the point where dangerous stresses begin. On the contrary, in the case of light bridges the live load is the controlling feature in fixing the safe stress unit, and here is just where the great discordance in practice comes in. The man is not born yet who can deduce a definite expression which will give even a rough approximation for determining the relation between the live load effect and its equivalent dead load. All the useful knowledge on this subject, worthy of the name, is that obtained from actual experience and observation on existing bridges. Of course the results obtained are more or less conflicting, but it is the best and only trustworthy information. By the nature of things this knowledge is purely empirical, and empirical formulas are notoriously dangerous tools to play with. Hence it is practically impossible to say, in the case of light bridges, where the point of safety ends and danger begins; there is necessarily a wide and uncertain interval. In such cases the unit stress adopted must be nothing more than an intelligent guess. The author's reflections are quite right when directed against present practice as applied to heavy bridges, but the writer is not at all prepared to admit the fairness of his remarks when applied to safe stresses for light bridges.

Mr. Katté. Walter Katté, M. Am. Soc. C. E.—As types of the long span lattice bridges on the New York Central Railroad, mentioned by Mr. Clarke, the writer has selected three representative spans. They were built before his connection with the road, and the following information was obtained from the records of the office.

Bridge No. 381, Mohawk Division, over Erie Canal at Schenectady. Mr Katté.
This bridge is a multiple intersection lattice riveted truss of 155 ft. clear span; depth of truss, 20 ft. It is divided into 23 panels of 6 ft. 8½ ins. each. The floor system consists of 9-in. I beams about 2¼ ft. apart, suspended from the truss bottom cords by four rivets of 1 in. diameter, in direct tension. It was built in 1866 and was strengthened in 1888 at a cost of $1 160 for labor and materials.

Bridge No. 578, Mohawk Division, over the Erie Canal at Canastota. The spans of this bridge carrying the two freight tracks Nos. 3 and 4 were built in 1864 and re-inforced in 1889. This span is 128 ft. in length with 20 ft. depth of truss. It consists of one center and two outside trusses, each containing 11 panels of 10 ft. each, and two of 8 ft. 11 ins. each. The greatest stress per square inch, as reported in the Railroad Commissioners' report of 1891 on bridge strains, is 11 300 lbs. per square inch.

Bridge No. 597, Western Division, over Erie Canal at Syracuse. This is a multiple intersection riveted lattice span, 130 ft. in total length, and 118 ft. clear span, with 20 ft. depth of truss. It carries three tracks supported by four trusses. The tracks are carried on transverse I beams which are suspended from the trusses by four rivets attached to each truss, the load being suspended by direct tension on the heads of the rivets. It was built in 1861 and was never strengthened until this year. It is now undergoing a process of re-inforcement. The stresses in certain members before strengthening were as high as 16 000 lbs. per square inch, yet it never exhibited any serious signs of weakness. The floor system is typical of bridges built at that date on the New York Central road, and would no doubt, if designed at the present day, call out considerable adverse criticism; nevertheless it has stood the test of 34 years of hard service, increased by being supported by weak and failing abutments, without requiring any of the rivets to be re-riveted, and the annual reports of the bridge inspector do not mention the detection of any loose or defective rivets.

The writer thinks there is no doubt but that Mr. Clarke correctly conceived the reason that this type of truss has shown so good a record, namely, "diffusing the strains over a large number of points rather than concentrating them on a few." From long observation of the bridges on the New York Central Railroad, he has arrived at the opinion that solidly riveted lattice girders unquestionably give the best, safest and most durable service for such constant and heavy traffic as that required by the New York Central system, and especially so in cases where the masonry support is not of the best. He considers the riveted lattice girder, with a modern solid floor track system, as the ideal railway bridge of the present day, up to spans not exceeding 200 ft. Beyond that length of span, economy indicates the desirability of using pin-connected tension web members and lower chord bars.

Mr. Pegram. GEO. H. PEGRAM, M. Am. Soc. C. E.—It is the prime duty of the engineer to save money in maintenance and construction, but it is his highest obligation to insure safety, and to the conscientious engineer a proper balance of these conflicting motives causes a great deal of anxiety. In designing a bridge, the specification of unit stresses enables him to proportion the parts with exactness. It would now seem wise to fix the limiting stresses governing the life of the bridge, which should be applied with the same precision in deciding upon its removal, and it is only by a consensus of opinion that these limiting stresses can be fixed.

The writer's observations agree with those recorded in the paper in the extent to which material has been overstrained with apparent safety. He has known numerous cases where wood has been used continuously in stringers at a stress of over 2 000 lbs., and where iron has been used, particularly in Howe trusses, at a stress of over 20.000 lbs. He is inclined to agree in the limiting stresses which the paper suggests may be used, but these can hardly be expressed as a definite percentage more than the bridge was designed for, because the stresses used in designing, as specified by different engineers, are not uniform.

The paper naturally brings up the question: What stresses shall be used in designing to secure proper length of life in a bridge? Iron bridges have had unreasonably short lives in the past on account of the rapid increase in the live loads. In future designs, special consideration should be given to this fact. The heaviest engine weight on the Union Pacific system increased over 60% in the ten years from 1881 to 1891. It hardly seems possible, however, that the heavy engines and trains now specified can be increased 50% on account of the limitations of cross-section.

The question raised as to the effect of a moving load compared with a static load must remain undecided until more experiments have been made on structures in service to determine the actual stresses in the various members and parts of members, and the writer inclines to the opinion that such experiments will modify considerably existing ideas with respect to details as well as to the comparative effects of live and dead loads, for it must be admitted that some of the "flimsy constructions" have given better service than some of the more rigid and confidence-inspiring bulky constructions.

A large gusset plate covered with rivet heads has an appearance of strength not always realized. Similarly, the writer knows of several cases in which light intermediate posts and bars of trusses have been bent by being struck, and the bridge saved from destruction by their bending out of line, gradually arresting the load by their deflections, whereas a large end post has been broken short off.

While improved materials and methods of manufacture might justify an increase in the unit stresses used in designing, engineers would

probably not agree to much increase, and wisely, because the unit stresses used 20 years ago were founded on experiments, and the conclusions drawn have been confirmed by subsequent experience. The evolution in specifications has been largely in the line of requiring more labor to produce practically the same results. The writer thinks it will be generally admitted that safe limiting stresses for a dead load, assuming wrought iron as the material, are 15 000 lbs. tension, and 12 000 lbs. compression, reduced by the usual Rankine column formula. Mr. Pegram.

In this connection the writer would indorse using one formula for all compression members, regardless of end fittings, which practice is now becoming more usual, and would make this formula the Rankine-Bouscaren pin-end column formula. This formula is the most theoretical in form of any used; it was originally based on experiments and has been supported by all subsequent experiments, and the numerous published tables make its application as easy as any other.

The straight-line formula, which was introduced for simplicity of application, seems to have opened a field for originality in the variation of constants, which has been so assiduously cultivated that it is impossible to make tables which will be of general use, and some of the modifications lead to absurd results.

It is so much more difficult to produce accurate fittings with square bearings, that in most cases the members are really weaker than those with pin bearings. Particularly is this the case in top chords where the camber must be taken out of the bridge before the bearings are theoretically square. It must also be remembered that a top chord will not stand as much proportionately in a testing machine as an intermediate post, because the former contains plates slightly curved between the rivets in the process of manufacture, while the latter is composed of channels which do not have this initial distortion.

If it were possible to be certain of the live load stresses, the unit stresses could be made the same as those specified for the dead load, but with the doubt existing in respect to impact and past experience in the increase of the loading, a margin must be allowed, based largely upon opinion. It is certain that the mass through which the live load stress is distributed will lessen its injurious effects, especially those caused by unbalanced parts, low joints, etc., and as the mass is less in short spans than long spans, it would seem reasonable to use the old method of making the impact addition decrease with the length of span. The writer would suggest an addition to the live load for impact varying from 50% at 15 ft. length of span to 10% at 100 ft., this percentage being added to the stresses produced from the loading and the sum considered as equivalent to a stress from static load. This is substantially the best practice of 20 years ago with a reasonable addidition for increased speed. It would then seem wise to allow a margin

Mr. Pegram. of 50% for future increase in the live load, so that in designing a bridge the dead load stress would be taken as given above, and the live load with impact added at 10 000 lbs. tension and 8 000 lbs. compression reduced by the column formula. Where the total stresses in a bridge exceed those specified for dead load, the bridge should be replaced.

These limiting stresses may seem conservative, but it must be considered that it is necessary to keep within the elastic limit, and the distribution of stress through a member or among several members is not uniform; posts may bear on the corners only; of two diagonal ties, one may carry all the load or one may be injured. The writer recalls several cases where the margin of strength has enabled a bridge to carry traffic after accidental injury until repairs could be made conveniently, and the savings of expense in such cases are often very large; but the most cogent reason is that the specified limits are generally exceeded.

For the purpose of comparing the unit stresses used in different specifications with those herein proposed, the following table has been prepared, the stresses given being those obtained by dividing the total stress in a member from dead and live load considered as static, by the section obtained under the specifications. The specifications taken, which the writer happens to have at hand, are, those published in 1890 by G. Bouscaren, M. Am. Soc. C. E.; those published by Theodore Cooper, M. Am. Soc. C. E., in the same year; the 1887 specifications of C. C. Schneider, M. Am. Soc. C. E., and the author's specifications, dated 1894. Mr. Schneider's later specifications are for soft steel, in which higher stresses are apparently taken than were previously used in wrought iron, and are therefore not used in comparison.

	Bouscaren.	Cooper.	Schneider.	Greiner.	Proposed.
Tension. Chords—					
200-ft. span	10 000	9 600	10 110	10 000	11 250
500-ft. span	12 000	10 667	11 070	11 225	12 000
Compression. Chords—					
200-ft. span, 30 radii	7 500	8 520	8 900	8 230	8 580
200 " 45 "	7 500	7 980	8 310	7 750	8 100
500 " 30 "	9 000	9 467	9 740	9 450	9 156
500 " 45 "	9 000	8 867	9 100	8 920	8 640
Intermediate posts—					
200-ft. span, 65 radii	5 670	5 090	7 180	6 630	7 140
200 " 130 "	3 640	2 080	4 360	4 360	4 600
500 " 65 "	6 400	5 600	7 810	7 725	7 570
500 " 130 "	4 110	2 290	4 740	5 450	4 880
30-ft. Plate girder—					
Tension	7 116	8 000	8 013	7 963	7 550
Compression	6 508	*7 320	*7 333	7 050	*7 000
60-ft. Plate girder—					
Tension	7 360	8 000	8 570	8 355	8 550
Compression	6 560	*7 040	*7 500	7 400	*7 640

*Same gross section as tension flange.

The increase the author shows in the rolling load, and therefore in the stresses to which bridges have been subjected, has been very general. In the face of such facts, the microscopic refinement to which calculations are carried seems absurd, and the waste of time caused by elaborate specifications seems inexcusable. Particularly is this the case in the persistent use of the engine wheel diagram. It was shown in a discussion of the paper* by Joseph M. Wilson, M. Am. Soc. C. E., on "Specifications for Strength of Iron Bridges," that a uniform load in combination with a single concentrated load would give the stresses obtained by the engine loading with practical exactness. Mr. Pegram.

It was further shown† in a discussion of the paper by Theodore Cooper, M. Am. Soc. C. E., on "American Railroad Bridges," that this equivalent loading could be made to include the effects of impact, thus abridging in a large measure the labor of calculations. It would seem as if the use of such equivalents, or the old method of using equivalent uniform loads, might be used with advantage. The actual engine loading must, of course, be the basis for any such equivalent, and might reasonably be stated in the specifications for the information of non-experts as the loading designed to be covered.

With respect to stresses in wood, consideration must be given to the fact that while wood does not lose strength, with age, to resist gradually applied loads, it loses fiber and ability to resist shocks. Some years ago the writer made some experiments on pine of different ages, and found that while the pieces gave practically equal resistances under gradually applied stresses in the testing machine, they were broken by the same weight falling from the following average heights: 13 ins. for new seasoned lumber; 10.6 ins. for lumber in service four years; 6.7 ins. for that in service nine years. Engineers are so accustomed to a large margin of strength in wooden structures, which are so often overstrained, that it hardly seems wise to make much increase in the unit stress. Although the writer believes the limits given in the paper might be safely used, he considers a unit stress of 1 200 lbs. per square inch for bending in yellow pine about right in designing new work, and a limit of 1 800 lbs. in removing old work.

R. Montfort, M. Am. Soc. C. E.—A bridge designed in accordance with the best modern practice would undoubtedly carry safely and without injury to itself, loads producing strains in its members 50% to 65% in excess of those for which it was originally proportioned. It has been the writer's custom to consider as safe a truss bridge of 125 ft. span or over, so far as the lower chords and main ties are concerned, if the strain per square inch in them does not exceed 15 000 lbs. Unfortunately, however, the details of old bridges are very rarely of equal strength with the bodies of the principal members, and, therefore, they Mr. Montfort

* See the *Transactions* of the American Society of Civil Engineers, Vol. XV, p. 474.

† See the *Transactions* of the American Society of Civil Engineers, Vol. XXI, p. 575.

Mr. Montfort. determine the life of the structure in a very large majority of cases. Thus, cast-iron joint boxes are found in many old bridges, while in a few still remaining, entire compression members made of the same material are found. The fact that a bridge has cast-iron joint boxes or members made entirely of cast-iron would not of necessity demand its condemnation, yet their existence, especially if poorly designed, might lead to its condemnation while the strains in the bodies of the principal members are well within the limit of safety. The writer has seen a cast-iron top chord removed from a 200-ft. span, which had been in service about 20 years and had carried loads more than 50% greater than that for which it was designed, show, when broken for the scrap pile, a thickness of metal on one side of less than $\frac{1}{8}$ in., while the opposite side was $1\frac{5}{8}$ ins. thick, due to the shifting of a short portion of the core used when making the casting and which was not discovered, although test holes were drilled close to the point where the displacement occurred.

Again, want of rigidity or of proper lateral bracing should be considered when passing upon the safety of a bridge. In most old structures the attachment of the laterals will be found defective and not sufficient to develop their full strength, while in others little or no provision is made to transfer the stresses from the lower lateral rods of through bridges into the bottom chords, or make other proper provision for the component of the stress in these rods now usually transferred into the chords.

The fact that until 20 years ago it was almost the universal custom to make eye-bars by welding the heads to the bars should not be overlooked. While the welds were generally made with great care, and the bars afterward tested to a strain of 20 000 lbs. per square inch, yet there have been numerous cases of failure under service in these welds. The writer has seen broken bars which showed only one-third of the section perfectly welded, yet they had withstood a strain of 14 000 lbs. per square inch under service for 15 years before breaking. He has never known a principal member of a bridge to fail in its body where a serious flaw did not exist. All of the failures of tension members that he has known of took place in bad welds or the minor parts, such as the smaller system truss rods, floor-beam suspenders and floor-beam hangers. He has seen truss rods with screw ends break in the screw when the figured strain on a section taken at the base of the thread was less than 12 000 lbs. per square inch, but this was probably due to the imperfect bearing of the nut. Floor-beam suspenders and hangers in old bridges fail probably more often than any or all of the other parts taken together, and frequently under extremely low strains. He has in mind three failures of hangers where the figured strain was only 7 000 lbs. per square inch, and all took place in similar spans and within a period of one year, the structures having been in service for 14 years before a break occurred. These hangers were of the usual

U shape, with screws and nuts, but there was only one hanger at each joint; the tendency of unequal strain, which is so likely to occur where two hangers are used, was, therefore, much reduced. The fracture occurred in all cases in the center of the hanger directly over the pin, where there was a full section and no weld. The hangers were made of 1½-in. round iron and rested directly on rough circular castings, which rested in turn on the pins. The failures were probably due to the imperfect bearing between the rough round hangers and the rough castings on which they rested, combined with an inferior quality of iron. Mr. Montfort.

J. P. Snow, M. Am. Soc. C. E.—The consideration of unit strains alone is not sufficient to determine whether or not a particular bridge should be renewed. In fact the units used by bridge engineers in designing new work are not derived from the scientific consideration of laboratory tests on materials, but from the study of structures in service. How else have they become reduced so much from what obtained 20 years ago? What else has brought down the units for steel construction from the estimates used when it first entered largely into bridge construction? Let it be supposed that a man is endowed with all the technical education that the world can bestow, except a knowledge of practical bridge designing; he shall be acquainted with all the experiments and tests that have ever been made on iron and steel; he shall have at hand all the multitude of typical engine diagrams and classes X, Y and Z loadings that locomotive builders have created or bridge designers evolved; but shall never have seen a specification, a set of plans, a bridge, or have any knowledge of the unit strains usually employed. He shall be called on to design a bridge, and by some unexplainable chance shall decide on a truss and floor system like the best modern development; in fact he shall be fully equipped for a full-fledged designer except that he shall have had no experience or knowledge of prevalent practice. Does any one think that he would adopt a scheme of unit strains that would produce a satisfactory bridge? He would adopt strains based on the elastic limit probably, but they would be far above what is considered good practice, very likely about twice what is generally used. Mr. Snow.

What, then, is the basis of the present units? Examinations of structures in service unquestionably. Any one who has seen a light bridge writhe and sway under a heavy train will understand why unit strains must be kept low. Probably the great majority of engineers copied their units from recognized authorities or were brought up under engineers who so copied them; but without question these low units originated with some one who studied his bridges in service and gradually lowered the units until a satisfactory structure was produced. The experiments on fatigue and similar subjects have had an effect in modifying the units used in the various parts of a truss, but

Mr. Snow. are not the basis on which the fundamental unit rests, and is it not just as scientific to experiment with a bridge as a whole as with little strips of its individual parts? The experiment need not be carried to rupture in order to ascertain whether or not it is satisfactory.

In order that a bridge may be satisfactory it must remain rigid under trains and deflect but very little. These conditions can only be obtained by using low unit strains. The practice (more markedly prevalent some years ago than now) of using much higher units on steel than on iron has always seemed inexplicable to the writer. Certain units, like shearing of rivets, and bearing on rivets, pins and rollers, may well be high in steel; but putting tension and compression (the strains that give rise to deflection) high for steel and low for iron when providing for a live load and calling the resulting design satisfactory seem illogical. The theorist may say it is right because the units compare with the elastic limits, but the practical man says it is not right because a light steel bridge is no stiffer than a light iron one; in other words, the units do not compare with the moduli of elasticity. Bridges are not built to resist breaking alone. It takes a very poor bridge to fall. They will become unsatisfactory and give rise to wholesome fear a long time before they will fail utterly. Units should be selected largely on the basis of resisting deflection. A girder having a generous ratio of depth to span may well be designed with higher units than a very shallow one.

The life of a bridge depends in large measure upon its rigidity. The calculated unit strains assist greatly in directing attention to weak parts and connections, but the limit between reasonable safety and danger cannot be fixed absolutely by the strain sheet. Study of the action of a structure under strains must be combined with a study of its strain sheet in order to reach a logical and defensible conclusion. Cases II and IV cited by the author show this; for, whereas an over-strain of 141% in the plate girder did not condemn it, an over-strain of 144% in the Bollman truss did. The 3% excess could hardly have been the determining factor. Cases are also mentioned in the paper of main rods in Bollman trusses snapping short off, showing that they had been strained beyond their elastic limit many times. Is it not the lack of rigidity in this class of bridges that allows oscillation which produces strains beyond the computed amount? Vibration, or "structural motion" as George H. Thomson, M. Am. Soc. C. E., calls it, is surely an enemy to the life of a bridge.

It has been the experience of the writer that loosely built bridges, like the early pin structures, become unsatisfactory and call loudly for renewal, while others of more rigid type stand satisfactorily, although the strain sheets of the latter class may show the larger unit strains. On the Boston and Maine Railroad the same tension unit is used in new work on built plate girders as on eye-bars, and 10% more on rolled

beams. Although this is not in accordance with accepted practice in certain high places, still, in the opinion of the writer, the beams and girders then make better and more serviceable bridges than do the trusses. Mr. Snow.

The units suggested by the author as the highest permissible for iron bridges seem to be on the safe side. Certainly in the case of rolled beams and plate girders it is likely that higher strains would not cause trouble if the track is kept in perfect condition. However, it does not pay to carry a bridge too long. If it is known that it must come off in a few years at best, the saving of interest on its cost is not to be considered beside the feeling of security that a new structure gives.

Timber stringers of long leaf pine are reasonably satisfactory with below 1 800 lbs. fiber strain. That is the limit at which such bridges are classed as unsatisfactory on the Boston and Maine Railroad, but they are by no means condemned. When strained to 2 200 lbs. they will carry the load, but will deflect disagreeably; 1 200 lbs. is the unit used in designing. For some reason both timber and iron will bear in a satisfactory manner a higher fiber strain when acting transversely as a beam than when acting longitudinally as a strut or tie.

The remarks of the author regarding poor track are very pertinent. A light bridge will last much longer with a perfect track upon it than if the rail is light, joints poor and alignment bad. This is another reason why the strain sheet must not be the only guide in condemning bridges. If a structure has been shaken 40 times a day for a series of years on account of a poor track, the figures will not show it, but the bridge itself will. Occasionally it becomes the disagreeable duty of the bridge engineer to condemn a bridge when the rest of the world think it good enough, but it has been the experience of the writer that generally the complaint comes from the operating or maintenance departments that a bridge is unsatisfactory, and it behooves the engineer to show cause why it is all right.

While the writer agrees with the safe limits of strains fixed in the paper, yet he believes there are few cases in which they will be the determining factor in deciding whether or not a bridge is safe.

THEOD. G. HOECH, M. Am. Soc. C. E.—In 1872 Dr. Strousberg, a well-known German railway organizer, erected a bridge of six spans near Forst, Germany, the wrought-iron superstructure coming from his own shops. After the railway was purchased by the Prussian government a thorough examination revealed numerous cracks as deep as 3 ins. in the chord plates of the pony trusses of the bridge. Some of these had been obviously patched up during construction. The bridge was replaced in 1893 with a new one of similar design, and the worst part of the old superstructure was carefully removed to properly arranged supports and tested to destruction.* The result was surprising. Mr. Hoech.

*See the *Zeitschrift für Bauwesen*, 1895, pp. 289–316.

Mr. Hoech. Under a uniform load of 6 759 lbs. per linear foot of track, the upper chords in one of the middle panels bent aside, the skew of 58° increasing the bending strains after deformation, and the pony trusses of the 99-ft. span settled upon the timbers piled 1 ft. underneath. The bridge withstood a stress of 38 000 lbs. per square inch in the chords, and failed because the horizontal lattice with tie plates between the halves of the chord members did not develop the full strength of the H section. The breaking test proved that a well-designed Schwedler bridge would have withstood much greater strains than those caused by the traffic, despite the old cracks, crystalline spots and slag flaws in the wrought iron. The surface of the metal was free from rust, although only ordinary care had been exercised in maintenance; no rivets had been broken, and the cross-beams and stringers, with their connections, were not damaged. It is worth noting that the vertical bending of the loaded trusses, when in service, did not indicate the fatal weakness of the upper chords.

Mr. Schneider. C. C. SCHNEIDER, M. Am. Soc. C. E.—The writer thinks good judgment was shown in the paper in making such a radical distinction in the permissible strains allowed for new bridges, where provision should be made for future contingencies, and the strains which are considered the limit of safety for bridges in actual service.

However, he does not agree with some of the author's statements and conclusions. For instance, he does not agree "that it is positively certain that the method of figuring stresses from concentrated wheel loads causes strains so far above what really exist as to almost class this method as unscientific." The author states that there can be no concentrated loads when there are stiff rails, etc. While it is true that there are no real concentrated loads such as are assumed in calculations (the loads on a bridge being distributed over two or three ties), this small distribution, however, will only materially change the effects of concentrated loads in exceptionally small beam or plate girder deck spans. On a 5-ft. span the bending moment may be reduced 25% to 30%, while in a 20-ft. span this reduction will be only about 5 per cent. On deck plate girders of larger spans, this distribution has very little influence, as the centers of application of the loads, as well as the end reactions, remain the same. This distribution, caused by the rails, would therefore only round off the angles of the polygon representing the moments produced by concentrated loads. In plate girder bridges with floor beams and stringers, as well as in truss bridges, the static effect of the loads will not be changed by this distribution, as the loads are distributed by the stringers over the length of one panel and concentrated at the panel points. It is therefore evident that the strains figured from concentrated wheel loads are not so very much above what really exists as to class this method as unscientific.

Mr. Schneider.

The author seems to take it for granted that the prevailing opinion among bridge engineers is that a live load produces twice the strain that is caused by a dead load. The writer, however, believes that there are not many who hold that opinion. While it is generally admitted that a suddenly applied load produces about twice the strain of the same load at rest, an engine and train moving over a bridge is by no means a suddenly applied load. Assuming the average speed of a train to be 60 ft. per second, it would take two seconds to fully cover a bridge of only 120-ft. span with a live load. This certainly cannot be considered a suddenly applied load. The fact is, that the load of a moving train is applied gradually, one pair of wheels at a time. The theory of the suddenly applied load would therefore apply approximately to very small spans only, which receive their maximum load from one pair of wheels, which would generally be spans of 5 ft. or less.

The paper further states that "no one can deny that a stress less than the elastic limit for dead loads, and less than one-half the elastic limit for live loads, must be used in considering whether or not a structure is safe." The writer begs to deny that. The static effect of a live load *per se* is the same as that of a dead load, depending upon the amounts and distribution of the loads only. If a train were moving over a bridge at a very slow speed, the strains in the various parts of the structure would be approximately the same as the static strains calculated from the same loads, with the exception of framed structures, wherein the members, on account of the deformation of the system, receive secondary strains in addition to the direct strains, which, on account of their complexity and the labor involved in calculating them, are generally not considered, due allowance being made either in the usual margin allowed for safety or by adding a percentage to the calculated strains for that purpose. The dynamic effect of a live load, commonly called impact, depends upon the conditions under which the live load is applied. The conditions which affect the impact are the conditions of the track, the dynamic action produced by the deflection of the bridge, the action of insufficiently balanced drivers, the reciprocating motion and vibration of the machinery, and the velocity of the train.

As the static and dynamic effects of a live load depend each upon such entirely different conditions, it seems more rational to consider the effect of each separately and distinctly, in order to arrive at a more scientific solution of the problem of determining the safe working strains in railroad bridges. As the internal stress in a member of a structure is proportioned to its elongation or reduction, it is evident that it makes no difference, as far as the resistance of the material is concerned, whether this stress is produced by the weight of the structure, by the static effect of a superimposed load, or by the dynamic

Mr. Schneider. effect of a moving load. If a permissible stress per square inch is safe for one strain, it is safe for all strains. If, therefore, the dynamic effect of the live load could be determined, there is no good reason why a uniform permissible strain should not be used for the static as well as the dynamic effects of the loads.

The author admits the dynamic effect of a live load; he also admits that the shocks or blows are more severely felt in the members which receive these blows suddenly and directly and which have little inertia of their own to resist and absorb them. In other words, he admits that the dynamic effect varies with the length of the span. He also recognizes this fact in his specifications by allowing a certain permissible working strain (15 000 lbs. per square inch for static strains), and adding 100% for impact for all-live load and smaller percentages as the dead load, or, in other words, as the length of the span increases; but prefers to disguise the fact by a modification of Launhardt's formula, which has no scientific basis, nor is it derived from data taken from actual experiments or verified by practical experience. However, the author claims that this formula is based on the foundation of fact; so the writer requests him to state the facts upon which this formula is based.

As the paper condemns all other methods of proportioning bridges, the writer wishes to point out some apparent inconsistencies in the Baltimore and Ohio Railroad specifications which need explanation. If Launhardt's formula, which is based on the theory of the fatigue of metal, is correct for permissible strains in tension or compression, the writer can see no good reason why it should not be used for shearing and bearing strains, as well as for combined strains. If the dead and live load strains on a member are equal, the paper allows a working strain 50% greater in tension or compression than if the strains were produced by live load only; but for the strains on the rivets in the same member it makes no such distinction, but specifies a uniform strain. If splices in two tension members of exactly the same sectional area were proportioned in accordance with the specifications, the one receiving strains from an all-live load, while in the other the dead and live load strains are equal, there will result two members of the same section, each one with a different number of rivets in the splice. The member in which the dead and live load strains are equal will have 50% more rivets in the splice than the one which receives live load strains only, yet in both members an equal amount of metal is to be spliced.

For members which receive combined transverse and direct strains, the Baltimore and Ohio Railroad specifications prescribe a uniform working strain per square inch, irrespective of the length of the span or the proportion of the minimum and maximum strains. If Launhardt's formula is founded on facts, then a uniform working strain is wrong.

Mr. Schneider.

There is either not enough material in these members if they belong to short spans, or a useless waste of material if they are members of very long spans, if proportioned in accordance with these specifications.

The writer does not agree with the author that experience with structures which have been working for some years under an increased traffic without injury furnishes the only means of solving the question of safe working strains for railroad bridges. No doubt these experiences are very valuable and instructive, but there is a more direct and scientific way of solving this question, and that is to determine the strains produced by the live load on the various members of a structure by actual measurement. The late Prof. Fränkel, of the Polytechnic School at Dresden, made some very valuable and interesting experiments on structures in actual service, with an instrument designed by him for the purpose of recording the actual strains in members of bridges produced by a moving train under different velocities. The writer has for many years persistently advocated a series of similar experiments to be made on American types of bridges, but has so far received little encouragement. He would be glad to cooperate with any railroad company that would furnish facilities for making these experiments. Large amounts of money have been wasted on useless experiments, such as the tests made on full-sized compression members, which have not even furnished data to develop a rational formula for calculating approximately their strength. As, at the present time, the knowledge of the dynamic effects of moving loads on railway bridges is limited, the subject seems important enough to receive some attention. The results received from carefully and conscientiously made experiments in that direction would be of great practical value.

As the paper denounces the method of considering the dynamic effect of the live load as guesswork, the writer feels compelled to say a few words in reference to the subject and to give his reasons for adopting this method. In 1877, the late Prof. Winkler, of Berlin, in a paper published in the *Journal* of the Austrian Society of Architects and Engineers, called attention to the unscientific method used at that time in proportioning bridges, and suggested that the dynamic effect of the live load should be considered in addition to its static effect as a more rational way than that of considering the static effect only and allowing a large so-called factor of safety to cover ignorance. He also proposed a formula for determining this dynamic effect, which, owing to the absence of any data at that time, was based more on scientific assumption than on practical experience. Later on, this subject was discussed by various other authorities, all of whom seem to agree on the correctness of the principle of considering the dynamic action of the live load in proportioning the members of

Mr. Schneider. railroad bridges. Some of the older specifications also called for a percentage to be added for impact to the stringers, floor beams and hangers. The writer believes, however, that he was the first to adopt this principle systematically, using a uniform permissible working strain in the "General Specifications for Railroad Bridges," published by the Pencoyd Iron Works in 1887. Attention was first called to the subject by Prof. Winkler's paper, and owing to his high standing in the profession and the regard for his ability held by the writer, the latter was prompted to give this matter serious consideration. He was convinced by the soundness of Prof. Winkler's arguments that this was the only correct method to use in proportioning railroad bridges. He then commenced to collect all the available information on the subject. Some very valuable data were obtained from Prof. Robinson's report on the vibration of bridges. At that time he learned that Prof. Fränkel had been making for several years experiments on railroad bridges for the purpose of determining the actual strains produced by the moving load under different velocities. In response to his request, Prof. Fränkel, whose experiments were commenced, the writer believes, in 1881, and continued until his death in 1894, kindly supplied him with all the data collected up to the time and furnished from time to time the results of subsequent investigations. They were made on a number of bridges of different spans of the European types. While the number and variety of these experiments were not sufficient to furnish data for making very close calculations of the effect of impact, they were sufficient to establish some facts which had heretofore only been assumed; they also furnished enough information to serve as a guide to determine in a general way the probable dynamic effect of a moving train which would not be exceeded in practice. These experiments have developed the following facts:

First.—That the dynamic effect of a moving train varies with the velocity of the train and the length of the span.

Second.—That members which receive the live load direct are most affected, while those that receive the strains from the live load indirectly are less affected.

Third.—The dynamic effect diminishes as the parts are removed from the members to which the loads are applied.

Thus, the dynamic effect of the live load would be most severely felt by the floor stringers, less by the floor beams and floor beam hangers; next would come the web system, and the least affected would be the chords. In a deck bridge, however, with the ties resting on the top chord, the top chord would be the part most affected by the dynamic action of the train. From the data on hand the writer deduced coefficients of impact, which he believed would be in excess of what really existed under the most unfavorable conditions, adding 20% for secondary strains, which percentage should not be exceeded

in any well-designed structure. Beam bridges and plate girders receive practically no secondary strains, but as very great refinements were not proposed, no distinction was made for plate girders. The writer hesitated a long time before using these coefficients for impact in actual practice; and not until he had applied this method to a great number and variety of bridges, to ascertain whether it would apply equally well to all kinds of structures and conditions, did he venture to embody this method in his "General Specifications for Railroad Bridges" and consent to have it published. These specifications have now stood the test of eight years, and have been used on ordinary trusses, trestle towers, arches, cantilever bridges, continuous spans and draw bridges and have in every case given satisfactory results. Mr. Schneider.

If a new bridge is rationally designed and low working strains assumed in accordance with the present practice in order to meet future contingencies, it should be able to stand a small increase of the live load in all its parts. In a pin-connected span with vertical posts and diagonal bars, counters are required in those panels where the bars receive strains in an opposite direction. If the bridge is proportioned in accordance with specifications based on Launhardt's formula, no counter is required in a panel where the live load compression is equal to the tension produced by the dead load. It is evident that if the live load is increased that the live load compression in that panel would increase in proportion and therefore require a counter; that is, while the chords and most other members of the truss would be strong enough to resist an increased live load, there would be some parts which would have to be reinforced or patched up in order to resist an increased load. If, however, this bridge is proportioned in accordance with the impact method, the live load strain is increased by a certain percentage; and consequently in this panel, where the static strains from the dead and live loads are equal, owing to the addition of the impact to the live load, a counter would be required, thus making provision for a reasonable excess of live load above that for which the bridge has been designed. In a bridge designed in accordance with specifications using the impact system, the strength of the details and connections will be in proportion to the main members, as the impact applies to all parts. Another advantage of the impact method consists in its simplicity; using a uniform working strain throughout, the permissible strains for compression members, shearing and bearing values for rivets, etc., can be tabulated and the work of proportioning the members of a structure thereby reduced to a minimum. If a system is correct it ought to be applicable to all cases and all kinds of structures. The impact method fulfils this condition. The writer is using his specifications for railroad bridges for highway bridges and buildings, by simply changing the coefficient of impact. For highway bridges he adds 25% to the live load strains, and for buildings discards

Mr. Schneider. the impact altogether. If future experiments should prove that the coefficients for impact adopted are too large, they can be easily corrected, which, however, would not affect the system, but would only involve a change of the numerical values of the coefficients.

Considering the cases cited in the paper, with a view of learning some practical lessons from them, the writer fails to see that these facts upset any of the well established theories of mechanics and statics, as the author claims. As the structures under these excessive loads did not show any indications of failure, it is evident that the maximum strains produced by the combined static and dynamic effects of the loads have been within the elastic limit of the material. The author has calculated the strains per square inch produced by the dead load and the static effect of the live load, which would be enough below the elastic limit to be considered safe if the live load did not also produce a dynamic effect, which was not considered. Had the actual strains been ascertained by measuring the elongation of the various members during the passage of one of the heaviest engines, the results would have shown at once the maximum strains the material was subjected to in every one of these cases, and would have enabled the engineer to judge whether these strains were enough below the probable elastic limit to consider the structure safe for the present conditions of the traffic. It would also have given the dynamic effect of the live load, which could be applied to the calculation of similar cases. If it is supposed that the actual strains in these cases had reached the usual elastic limit of 26 000 lbs. for wrought iron, there would be the following dynamic effects of the live load for the first three cases :

Case I, 26-ft. span	65	per cent.
Case II, 20-ft. "	60	"
Case III, 84-ft. "	62.8	"

The dynamic effects of the live load, as given in the Pencoyd specifications for the above cases, are 72, 74 and 79% respectively. The remaining two cases the writer has not been able to analyze, on account of insufficient data. These cases prove the statement he made before, that the coefficients for impact proposed by him are in excess of those that really occur in ordinary traffic.

For the purpose of determining the limit of safety in existing structures, the writer would suggest using the impact system as given in the Pencoyd specifications, except for plate girders, for which he would make a reduction of 20% in the coefficient for impact, as they receive practically no secondary strains, and allow a working strain of 22 000 lbs. per square inch, properly reduced for compression; and calculate the fiber strains in the flanges by the moment of resistance of the cross-section. If a working strain of 22 000 lbs. per square inch is allowed as the safe limit for existing structures, the bridges designed in accord-

ance with the Pencoyd specifications would be able to stand the follow- Mr. Schneider.
ing increase of live load :

25-ft.	span,	plate girders,	49 per cent.	150-ft.	span,	59	per cent.
50-ft.	"	"	51 "	200-ft.	"	62	"
75-ft.	"	"	53 "	300-ft.	"	70	"
100-ft.	"	"	55 "	500-ft.	"	80	"

J. E. Greiner, M. Am. Soc. C. E.—In the body of the paper the Mr. Greiner.
author endeavored to lay force upon certain points, and, in closing this discussion, these points will be considered separately and in the order of their occurrence in the paper.

I. "Owing to an increase in train loads and flimsy construction the life of railroad bridges heretofore has been scarcely 25 years, but the life of a bridge of good design will be determined by the care bestowed upon its maintenance rather than by tonnage or years, provided specified loads are not exceeded. If placed in the hands of indifferent and incompetent persons, there will be a greater certainty of its failure before than of its existence after 25 years."

Mr. Stanwood has presented some interesting illustrations showing the actual loss of section caused by corrosion due to neglectful maintenance, and their value as records of the actual amount of deterioration per year cannot be overestimated. Structures subjected to the same evil influences are met with on almost every railroad, and if they stand for a period of 25 years, the fact can be attributed mainly to the intelligence bestowed on their maintenance. There are effective means of protecting iron against such injury, and if some one of these means is not employed, the same results as pointed out will take place sooner or later.

Attention has been called to the satisfaction given by multiple system riveted trusses on the New York Central Railroad. Some of these bridges have for years carried a traffic much greater than was contemplated at the time they were built, and their great stiffness and durability are attributed to the fact that strains are diffused over a large number of points instead of being concentrated on a few. Mr. Snow states that it has been his experience that loosely built bridges, like the early pin-connected structures, become unsatisfactory, while those of a rigid type are doing their work all right, although strain sheets indicate higher unit stresses. This is a very general experience and has led to quite an extended practice of designing bridges so that they will have a small number of large-size members rather than a larger number of smaller members, thereby concentrating strains and metal instead of diffusing them. The author ventures the opinion, that had the bridges on the New York Central been of this character, they would have stood just as well as the bridges mentioned, and that their stiff members and firm connections may have contributed more to their rigidity than the fact that the metal and strains were diffused.

Mr. Greiner. II. "The avowed purpose, when designing a bridge, is to develop a structure which will safely carry the specified loads. If this structure will be perfectly safe under a greatly increased load, then, from an engineering sense, more material has been used than was necessary for the avowed purpose, and this waste cannot be considered good engineering."

Mr. Le Conte, while admitting that engineers are placing more material in their large bridges than is necessary for carrying the specified loads, will not acknowledge that any excess has been used in small spans. The author understands that Mr. Le Conte would approve of somewhat higher unit stresses than are commonly used for large spans, but would not endorse an increased stress in small spans. Considering the fact that bridges are designed so as to carry a future increase in traffic of a very indefinite extent, the author would not use higher stresses than specified. The waste in material is not because the bridges are built too strong for future emergencies, but because they are being built too heavy for their avowed purpose of carrying in safety a certain specified load. The low stresses usually employed have not been so commonly adopted, merely because experience has demonstrated them to be safe, but rather because experience has demonstrated that when a bridge is nominally designed to carry an 80-ton engine and tender it will very likely be called upon to carry 120-ton engines. Railroad companies are led to believe that they are getting structures proportioned for a definitely specified load, but they are paying for structures really designed for much heavier loads; in other words, stress sheets represent more than they profess. Would it not, therefore, be much better engineering practice to design bridges strictly for the maximum loads specified and take these specified loads from 50% to 100% in excess of immediate requirements? If this were done, unit stresses could be increased to the limits which experience justifies, and when the train loads have exceeded those for which the bridge was designed, it will have reached the end of its usefulness. Stress sheets would then represent all they profess, but not more, and railroad companies would be paying for what they expected to buy, but not for more. It is needless to state that such an innovation will not take place in the near future, owing to various prejudices, but it will be safe to predict that when the increase in locomotive and train loads ceases, as it surely will, engineers will then design strictly for the heaviest possible loads, or else they will be subjected to the alternative imputation of either specifying loads less than are in service, or of being extravagantly wasteful in the use of material.

III. "The condemnation of a bridge is a case for figures, experience and judgment, and involves a responsibility which should not be left to the unsupported opinion of the practical bridge man, or to the impetuosity of the theoretical alarmist."

Mr. Morison refers to a case where the calculated bending stress in the extreme fibers of bridge pins was found to be from 175 000 to

250 000 lbs., but when the pins were removed they were found perfectly straight. The stresses never existed. If such cases of calculated overstrain should be referred to many purely theoretical men, they would in all probability have ordered the bridge to be trestled at once, thereby involving a large amount of useless expense. Mr. Greiner.

Mr. Hazlehurst has known bridges to carry a heavy traffic for many years with entire safety with a unit stress that would horrify a man not familiar with, or one who was not accustomed to watch, the behavior of structures under heavy loads.

Mr. Yeatman calls attention to a bridge which carried for years, without ill effects, loads producing a calculated pressure on bearing area of rivets in web of floor beams of from 36 000 to 40 000 lbs. per square inch. The author knows of several plate girders, the rivets in which were strained equally as high without sign of weakness, and has met with two cases of box-beam girders, which failed by the rivets in the bottom flange shearing off bodily for a distance about equal to the depth of the beam, although the calculated stress was but about 60% or 75% more than allowed by common practice. These bridges had been examined by the author not more than three months previous to their failure, and to his personal knowledge there was nothing to indicate that the rivets would shear off. The bridges had also been inspected each month by the regular bridge inspector, and reports showed them in good condition up to the time of their failure. These bridges, therefore, while overstrained, gave no evidence of weakness before actual failure, and were no cause of uneasiness to maintenance officials. Mr. Snow says it takes a very poor bridge to fall. These were possibly poor affairs, although they fulfilled their mission, but they did not "become unsatisfactory or give rise to a wholesome fear a long time before they utterly failed."

Mr. Hoech calls attention to a bridge in Germany which, after 21 years' service, was found to have numerous cracks as deep as 3 ins. in the chord plates of the pony trusses of the bridge. The bridge was condemned, and when tested to destruction it withstood a stress of 38 000 lbs. in the top chords, despite the defects which caused its condemnation.

The failures of rods and hangers mentioned by Mr. Montfort are fully as instructive as any of the above, and the three failures of U hangers at 7 000 lbs., which broke directly over the pin, were similar to those mentioned by the author as having broken at 12 930 lbs. at the side of the pin instead of through the threads, where the section was larger. The author has met with many cases of broken hangers, but never any which broke at a less stress than the case mentioned in his paper, and the evidence given by Mr. Montfort will cause him to fix upon a lower limit than given. His experience with eye-bars also accords with that of Mr. Montfort; they usually break through

Mr. Greiner. the welds. Eye-bars which have been in service for over 20 years, and for a long period at a stress of 14 000 lbs., have stood without failure, although they were found to have cracks as much as ¾ in. deep on the outside of the head. When broken the heads are usually one mass of crystals. There is a case of failure of a Howe truss which may be interesting in this connection. The bridge was 140 ft. long, and was correctly designed for an 86-ton engine and tender, followed by a train load of 3 000 lbs. per foot, and was built during the end of the year 1888. In the meantime train loads had increased 25%, and, as there was much shifting over the bridge and it began to shake under this shifting, it was trestled at the center in the fall of 1893. The trestle bent was put in place by practical bridge men, without engineering supervision, and, in order that it would run with the stream, it was placed so that the cap would meet the chords at an acute instead of at a right angle. It supported the outer stick at a distance of 4 ft. and the inner stick at a distance of about 2 ft. from the center panel point. The weight of the dead and live loads was, therefore, transferred to the bent by means of a bending in the lower chord. The lower chord was composed of four sticks, 7¼ x 14 ins. During July, 1895, the lower chord pulled apart 2¼ ins., and an examination disclosed the fact that three sticks had practically broken in two, not at the splices, as would be supposed, but through the body of the sticks in the neighborhood of the panel point near the center of the bridge. The fourth stick was not broken, but had pulled a little at the splices. In addition to the direct breaks, there were cracks about some large knots in the outer stick, which indicated that it had been broken by transverse strains. In addition to the above breaks, the end bottom chord broke in the center of the first panel, which was due partially to the transverse strains caused by the floor system, which rested on the bottom chords, and partially to the fact that the timber bed-plate had been placed outside the line of action of the inclined end post. No accident resulted from these breaks. There was nothing in the appearance of the surface of the timber, which was white pine, to indicate that it was not in fairly good condition, and experience with the same kind of timber shows it usually lasts eight years or more before it becomes seriously unsound, and it was reported fair up to the time of the failure. An examination at the points of failure showed that the sections of the chords were either what would be called lifeless timber or were positively decayed.

Experience with this bridge should teach three good lessons:

First.—In placing a trestle or bolster under a Howe truss, or any other truss, it should be placed directly under the panel point, not on one side of the panel.

Second.—The fact that timber is only about seven years old should not cause an undue confidence in its condition. In this case the lower

chords were just as badly decayed in less than seven years as some other timbers are after twelve years. Mr. Greiner.

Third.—Bridges designed for regular traffic are not always fit for shifting purposes.

IV. "The terms 'safe working stress' and 'maximum stress' have no definite meaning, owing to the various ways engineers have of providing for the dynamic effects of live loads. Static stress can be figured, impacts must be guessed. When the two are combined to form what is called the total stress, the result is a guess and the actual figured stresses are obliterated. It would therefore appear better practice if engineers would preserve the stresses which they can figure and provide for dynamic effects by varying the unit stresses in the different spans and members inversely as the effects of shock vary."

In order to provide for the dynamic effects of moving trains, defects in material, for bulk in members, and to be a little safer than just safe, the author stated that "no one can deny that a stress less than the elastic limit for dead loads and less than one-half the elastic limit for live loads must be used in considering whether or not a structure is safe." Mr. Schneider says he can deny it. As stresses must be either less, equal to, or more than, a stated amount, and it is denied that they are less, it follows that they are equal to it or more. In order to sustain his denial it should have been shown that dead load stresses can be taken equal to or above the elastic limit, and live load stresses equal to or more than one-half the elastic limit. As this would be quite contrary to Mr. Schneider's practice, the author cannot understand the force of his positive denial, unless he means that by increasing the figured stresses by the addition of impact percentages he can use one common unit stress for both dead and live loads. He does this in his specifications, and while he uses a uniform unit stress approximately one-half the elastic limit, when this stress is separated into its two component parts, the one for the dead load and the other for the live load considered without additions for the impacts, there will result stresses which are below the elastic limit for dead loads and below one-half the elastic limits for live loads, a statement which has been denied. This will hold true for his limiting stresses also.

Mr. Schneider says that the formula used by the author disguises the fact that 15 000 lbs. per square inch is employed for static stresses, and 100% is added to the figured stresses for the dynamic effects of an all-live load. Is not the reverse equally true, or in other words, does not the 100% impact and a single stress of 15 000 lbs. simply disguise the formula $7\,500\left(1+\frac{\text{min.}}{\text{max.}}\right)$? It therefore appears that there is no practical difference in the results of Mr. Schneider's method and that used by the author. The one disguises the statement of the other, that is all; and since one represents the fact that stresses less than the elastic limit for all-dead, and less

[Mr. Greiner. than one-half the elastic limit for all-live, loads are used, the other recognizes precisely the same statement, and the denial of the author's assertion has not been sustained.

The author does not condemn all methods of proportioning bridges other than the one proposed by Joseph M. Wilson, M. Am. Soc. C. E., in his paper on "Specifications for Strength of Iron Bridges."* As was stated in the paper the different methods make no material difference so far as weight and strength of bridges are concerned when designed for the same loads. It is the lack of uniformity in unit stresses as used by different engineers which were condemned as confusing to those who have not the time or inclination to look into the allowances which have been made for impacts. In order to compare unit stresses, they must all be reduced to the same basis, and it surely would be more consistent practice if this result would take place. Few will deny that engineers have much to learn concerning the dynamic effects of moving trains, and while the experiments quoted by Mr. Schneider indicated certain results in a particular case and gave a basis upon which to make guesses as to the probable effects in other cases, can it be asserted that the percentages, no matter how intelligently worked out, are anything more or less than guesses ?

The author has been asked to mention the facts upon which the formulas used in his specifications are based. These facts are mentioned in the paper on page 299, and are substantially the same as those upon which most specifications are founded, the difference being that in the impact percentages to figured stresses, the sum is considered as the total stress, while it may or may not be the total stress, and in the proposed method the figured stresses are surely what they represent to be, namely, stresses produced by the loads considered as static.

The various methods of arriving at unit stresses may be mentioned in this connection as indicating the different means of obtaining the same ultimate results.

First.—Impacts for counters, hangers, floor beams, stringers and spans under 100 ft., and a unit stress the same for live and dead loads.

Second.—Impacts as above, and unit stresses for dead load 50% greater than for live load.

Third.—Impacts for all spans and all members, and a unit stress the same for live and dead loads.

Fourth.—No impacts and dead load stresses 100% greater than those for live loads.

Fifth.—No impacts and variable unit stresses.

All of the above methods are in use and each has its advocates. The author therefore should not be condemned because he has selected the method which appears to him as the most rational, and has inad-

* See the *Transactions* of the American Society of Civil Engineers, Vol. XV, p. 389.

vertently criticised other methods with the hope that more specifi- Mr. Greiner.
cations will in future follow those prepared by Mr. Wilson, so that when an engineer speaks of a strain or stress in a bridge it will be known what he means, and when he speaks of a safe working stress it will be generally accepted as meaning that no impact additions have been made to the figured stresses.

As Mr. Schneider has pointed out what he considers as inconsistencies in the Baltimore and Ohio Railroad specifications and has asked for an explanation, it behooves the author to make reply.

To be consistent, the Launhardt formula should be applied to shearing and bearing values of rivets, etc., but consistency is not a virtue in this respect. These values are taken low enough to meet the worst effects of a moving load They relate to details, and when details are strong enough to develop the strength of the members which they connect, a little more strength in some cases than just sufficient will not impair the safety of the structure, will add practically nothing to the cost of the bridge, and will avoid a refinement in calculations where such refinement will not justify the labor involved. In the case of the two members having equal section, the one being subjected to an all-live load and the other to equal live and dead loads, the two members would, as Mr. Schneider states, have a different number of rivets in their splices. Although the sectional area of the members may be the same, the figured stresses are different, and the rivets would be proportioned for the stresses, not for the area of section. There would be a sufficient number for the case of all-live load and more than sufficient for the live and dead load. Can any one seriously object to this when the excess pointed out happens only in extreme cases and involves so little material ?

The Launhardt formula is not applied to combined stresses in chords of bridges, because the author considers bridges having cross-ties resting directly on the chords unsatisfactory, except in riveted trusses of spans between the limits of 80 ft. and 100 ft., and owing to the lack of accuracy in determining the stresses, which are affected by deflection, splices and position of pins, he would prefer not to use this arrangement in larger spans. Between the narrow limits of 80 ft. and 100 ft. the unit stresses would vary to so slight an extent as not to warrant any change, and a fixed value in accordance with good practice was therefore assigned.

The comparison of the two methods, when applied to counters in truss bridges, is true enough from a theoretical standpoint, and the specifications should have stated that web members not subjected to counter strains by the specified load must be designed to withstand the counter strains caused by an increased load of 100%, or else that counters must be provided for one panel in addition to the requirements of the specified load. Practically no engineer in good standing

Mr. Greiner. would leave out counters in the panel where the live-load compression is equal to the dead-load tension, or where these opposite strains are so nearly the same as to cause doubt.

In comparing the author's specifications with his own, Mr. Schneider should remember that the former specifications were made to meet the conditions existing on the Baltimore and Ohio Railroad. They are special, not general, and have a limited application. On the contrary, Mr. Schneider's specifications are general, and are designed to meet the requirements of any case whatever, and therefore their consistency is with them a virtue. The Launhardt formula can also be consistently applied, should it be desired, and the author still believes it would tend toward more uniform practice should engineers vary the unit stresses instead of varying the figured stresses by percentage additions.

V. "The unit stresses of 750 lbs. for white pine, and 1 000 lbs. for Georgia yellow pine, as given in text books since Lanza's experiments, are absurdly low, because good and safe practice sanctions much higher stresses."

When the author mentioned absurdity in connection with working stresses applied to rotten timber, he did not mean that in fixing the sections of timber stringers it is absurd to consider and provide for future possibility of decay, as Mr. Bates has understood it. The absurdity does not refer to making a reasonable provision for future deterioration, but in advocating a fiber stress as low as 750 lbs., when good and safe practice has justified a much larger stress. Of course timber cannot always be renewed or strengthened on the first appearance of decay, but when decay has attained such an extent that the amount of sound timber becomes an uncertainty, it is time to renew it and it usually is renewed, no matter whether the figured stresses on the nominal section be 1 500 or 750 lbs. The primary defects in timber, such as knots, shakes and sap, as well as season cracks and decay, must surely be considered in proportioning timber beams as well as in fixing upon what limiting stresses should be permitted, and when advancing the statement that 1 600 lbs. for white pine and 1 800 lbs. for Georgia yellow pine are the highest limits that the author would sanction for sound timber, he purposely left the unsound timber to individual judgment. Safe limits can be fixed approximately for sound timber, but who can assign a safe working stress to rotten timber? There have been cases coming under the author's notice where timbers were carrying the regular traffic up to the time of their removal, but when taken from the track and turned over on their sides they broke from their own weight, the interior being a mass of dry rot.

VI. The cited examples of overstrained bridges appear to indicate either that an all-live load does not produce twice the strain caused by an all-dead load, or else the usual method of figuring stresses by wheel-load moments is wrong. The author does not claim that a study

of the examples cited upsets any of the well established theories of mechanics and statics. Is it a theory of mechanics that a load moving over a bridge is a suddenly applied load? Mr. Schneider has denied this to be a fact. Is it a theory of statics that the wheel-load moment method applied to a bridge is absolutely correct for the conditions existing? Mr. Schneider has shown that strains in small spans can be less than those figured by this method. He also shows that an examination into the first three cases reveals the fact that an added impact of from 60% to 65% would have strained these bridges up to the elastic limit, and since they were not strained to this limit the impacts were even less than given. The assumption that an all-live load does cause twice the stress produced by an all-dead load is made in all specifications which use the unit stress formula, $a = u\left(1 + \frac{\text{min.}}{\text{max.}}\right)$. If the elasticity be taken at 26 000 lbs. the value of u in the above formula must be not more than 13 000 lbs. in order that this limit may not be exceeded In applying the formula to the cases in question, it will be found that in every case the value of u exceeds the 13 000 lbs., and this indicates that the elastic limit had been passed; but as it really had not been exceeded, the author maintains that the examples tend to show that an all-live load does not cause twice the stress of an all-dead load, and that therefore the assumption is wrong, although it is wrong on the safe side. Mr. Greiner.

VII. Engineers have sifted the good from the bad in bridge designs, and while there are differences of opinion as to what is and what is not just a little better than good enough, there is no question that all will agree upon what is fairly good construction, and, as was clearly set forth in the body of the paper, this class of bridge only is under discussion. The safety of a poor bridge cannot be discussed in general terms, but the safety of a good one can be. There are, of course, many cases in which the actual stress to which each individual member is subjected cuts no figure because the design may be poor and too shaky for the increased loads. There are also many cases in which it becomes necessary to limit the unit stresses in main members, although the structure apparently is capable of carrying still greater loads. Details should receive individual attention, independent of the attention bestowed upon main members, and in considering them it should be remembered that the strength of the weakest member in the weakest bridge on the line of a railroad is a measure of the capacity of the traffic on the road. If a single floor beam connection should break and cause a wreck, it would surely be poor consolation to know that all other connections had ample strength. Uniformity of strength is the desideratum of bridge construction. The quantity of strength is more or less a matter of judgment, and in proposing formulas for the maximum working stresses, which should be allowed in main members of

Mr. Greiner. bridges of good design, the author had in view uniformity of strength in main members, and the details strong enough to develop this strength. It will be noticed that some engineers, in considering the safety of an old structure, entirely disregard impact, and will allow a uniform safe working stress about 50% in excess of what they would allow on a new bridge where impacts are considered. This assumption if followed out will place suspenders and light members of trusses in positive danger long before the limits fixed have been reached in the main members. The author, however, understands that those proposing these limits use them merely as guides for final judgment.

In order to make a fair comparison of the limiting unit stresses which are considered safe, satisfactory or advisable by those who have ventured an opinion, the author has figured the flange and chord stresses in plate girders and trusses up to 200 ft. span. The limiting unit stresses given were then modified where necessary, so as to represent the quotient obtained by dividing the actual calculated stresses by the area of section of the member considered. The bridges were figured for 86-ton engines followed by 3 000 lbs. per foot, and a dead load given by Mr. Pegram's formula for this class of engine, with the addition of 350 lbs. per foot for weight of ties, rails, etc. The sections were determined by applying the formula used in the Baltimore and Ohio specifications.

Table A shows limiting stresses, the percentage of increase of the limiting stresses over the unit stresses used in the design, and the percentage of increase of live loads over the 86-ton engine.

An examination of the table reveals quite a variation in the limiting stresses for plate girders, but a much better agreement in those for the trusses. Had the table been extended, the proposed unit stresses would have increased, as would also those of Messrs. Wilson and Schneider, while those of Messrs. Pegram, Hazlehurst and Yeatman would remain constant. The author knows that Mr. Montfort would not adhere strictly to 15 000 lbs. as spans increased in length, and would consider the limits fixed by the author as approximating the same as he would probably use in such cases. Mr. Wilson has not fixed upon the units given opposite his name, but merely considers them safe, without saying whether he would go higher or not. Mr. Pegram's proposed units are very low for plate girders, although he stated in his discussion that he was inclined to agree with the limits fixed by the author. Messrs. Bates and Snow also seem to believe that the stresses proposed by the author are on the safe side when the conditions are as stated.

The limiting stresses for timber beams appear to be considered about right, although some would use 1 500 lbs. for white pine, and others would go as high as 2 000 lbs. for yellow pine, it being understood that the timber is perfectly sound.

TABLE A.—Limiting Tension Stresses in Main Members of Bridges.

	Plate Girders.									Trusses.					
	30 ft.			60 ft.			90 ft.			150 ft.			200 ft.		
	Limiting stresses.	Percentage of stress increase.	Percentage of live load increase.	Limiting stresses.	Percentage of stress increase.	Percentage of live load increase.	Limiting stresses.	Percentage of stress increase.	Percentage of live load increase.	Limiting stresses.	Percentage of stress increase.	Percentage of live load increase.	Limiting stresses.	Percentage of stress increase.	Percentage of live load increase.
Proposed	14 040	75.5	88.2	14 690	72.0	92.3	14 880	69 6	93.2	15 560	58.1	84.5	15 950	56.5	88.1
Wilson	11 420	42.8	50.0	11 870	39.0	50.0	12 05 0	37.4	50.0	13 225	34.4	50.0	13 460	32.1	50.0
Pegram	10 840	35.4	41.5	12 050	41.1	52.7	13 140	49.8	66.7	15 000	52.4	76.2	15 000	47.2	73.6
Hazlehurst	13 500	68.7	81.6	13 500	58.1	74.5	13 500	53.9	72.1	15 000	52.4	76.2	15 000	47.2	73.6
Montfort	15 000	87.5	97.8	15 000	75.6	97.0	15 000	71.0	95.0	15 000	52.4	76.2	15 000	47.2	73.6
Yeatman'	15 000	87.5	97.8	15 000	75.6	97.0	15 000	71.0	95.0	15 000	52.4	76.2	15 000	47.2	73.6
Schneider	11 510	43.9	51.7	12 860	50 6	64.8	13 400	52 8	70.5	14 430	46.6	67.8	15 120	48.4	75.5

Mr. Greiner. In closing this discussion, which has been carried to an extent not anticipated, the author desires to express his appreciation of the free and candid opinions advanced. The discussions have been of a practical nature, and the records of what bridges have stood and what they have not stood will surely be a future assistance to those who have questionable bridges to look after, and who are undecided as to what action should be taken. A good rule in case of serious doubt is to give the traveling public, not the bridge, the benefit. Iron is cheap when compared with human life.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

765.

Vol. XXXIV.—November, 1895.

THE DISCHARGE OF THE MISSISSIPPI RIVER.*

By WILLIAM STARLING, M. Am. Soc. C. E.

READ NOVEMBER 6TH, 1895.

Difficulty of Ascertaining the Discharge of a Great River.—To ascertain with accuracy the discharge of a large river is at all times a delicate and difficult business. When the stream is of enormous dimensions, when the current is very swift and irregular, when the bed is very rough, the water full of boils, eddies and cross-currents, when the banks and bottom are highly unstable, and are suffering considerable mutations from day to day, then the difficulty of the task is so greatly magnified that engineers who have undertaken such a work have more than once been tempted to give up, in despair of securing even a reasonable approach to precision, and many have abandoned the effort and have been contented with approximations, not pretending to come nearer than within ten per cent. or so of the truth.

The Extreme High-Water Discharge of the Mississippi Has Never Been Measured.—It has never been found practicable to ascertain by direct observation the extreme high-water discharge of the Mississippi,

*The Discussion and Correspondence on this paper, part of which was presented at the meeting of November 20th, 1895, will be printed in a subsequent number.

because the stream has never been confined between its banks at such a stage. There are some portions of the alluvial valley of the Mississippi where the river has never been restrained by levees at all. In the part which is so controlled, the levee system is still so far from complete that in great freshets the levees have broken at one place or another before the top of the flood has been reached. The measurement of the discharge at such times has been only partial, and independent estimates have been made of the losses through crevasses, escapes over unleveed banks, etc., which estimates are usually very rough and unsatisfactory.

For a clear understanding of what is to follow, it will be necessary to have some knowledge of the topography of the Mississippi Valley.

Topography.—The alluvial plain of the Mississippi begins a short distance above Cairo, and embraces small portions of the Ohio Valley and that of the united upper Mississippi and Missouri Rivers. The Mississippi proper, or lower Mississippi, then, occupies a wide valley or bottom, lying between two nearly parallel ranges of tertiary hills, usually capped with loess. This plain runs in a direction from a little east of north to a little west of south, and occupies a distance, measured by parallels of latitude, of 7° 51′, or about 544 miles. By an air-line it is almost exactly the same. The course of the river from Cairo to the head of the passes is about 1 060 miles.

The river does not run through the middle of its valley, but like other streams skirts the hills, first on one side and then on the other. After leaving Cairo, its tendency seems to be southward, but it is held in check by the bluffs on the left bank, and consequently hugs their base, with the exception of a few small basins, till it reaches Memphis. The hills on the right or western bank are consequently far from the river. The valley here expands into a large area known as the Saint Francis Basin, because it is drained by the river of that name.

The Saint Francis Basin.—The bluffs which border the western limit of this territory meet the Mississippi at Helena, just below the mouth of the Saint Francis River. There are no gaps in this range of low hills (called Crowley's Ridge), and all the water which escapes over the banks of the Mississippi into the Saint Francis Basin returns into the main river, except such as is permanently retained in the swamps and lakes or is evaporated or lost by leakage.

There are several secondary ranges of hills in the Saint Francis

Basin, which play an important part in the phenomena of overflow. The first of these extends from Commerce to New Madrid, in Missouri. This ridge is so high and has so few gaps that it almost isolates the small part of the great basin cut off by it from the rest; the water that leaves the main river above New Madrid nearly all returns at or above that point.

The other ridges are only of local consequence.

The Saint Francis Basin exercises a great disturbing influence on the discharge of the Mississippi at high stages. A quantity estimated at 250 000 or 300 000 cu. ft. per second is abstracted from the river between New Madrid and Memphis, and returns, sooner or later, between Memphis and Helena. The basin is of great dimensions, containing about 6 000 sq. miles, exclusive of the New Madrid Basin, and its action as a reservoir is very complicated, and is still imperfectly understood.

The Yazoo Basin.—On the left bank, below Memphis and extending as far south as Vicksburg, is the great Yazoo Basin, formed by the Mississippi and a semi-circular range of hills. It is completely lined by levees, and exercises no disturbing influence except when crevasses occur. It is to be observed, however, that in 1882, 1883 and 1884, it was only partially protected by levees, and that enormous volumes of water escaped over the banks and through breaches in the dikes, returning at Vicksburg, where the Yazoo River enters the Mississippi, draining the entire basin. In 1890 several great crevasses occurred, by which the regimen of the river was considerably altered. The area of this territory is 6 648 square miles.*

The White River Basin.—Below Helena, on the right bank, there is the small but important White River Basin, containing 956 square miles. This area is formed by a recession of the hills to a distance of 13 or 14 miles. Through it enter the White and Arkansas Rivers, which interosculate at their mouths, and discharge mostly through a common orifice, the old mouth of White River, though the disused mouth of the Arkansas sometimes delivers a very considerable volume. The two mouths are about 9 miles apart, the mouth of the Arkansas being the lower. Several small streams contribute to drain this basin, and empty into the Mississippi above Arkansas City. The principal of these is Cypress Creek.

* These and the figures for the other basins are taken from the valuable publication, "The Mississippi from St. Louis to the Sea," by J. A. Ockerson, M. Am. Soc. C. E.

The White River Basin has no well defined natural boundary on its southern side, but is joined to the Tensas Basin. An artificial limit has been given to it, however, by the erection of a high and strong embankment along the right or southern border of Cypress Creek, extending to the uplands, which are here only about 5 miles in a straight line from the river. Though relatively high and pre-alluvial, these lands, like some of the ridges in the Saint Francis Basin, are not entirely above overflow. In times of great floods from the Arkansas and White Rivers, considerable volumes of water have been discharged around the end of the levee, as will be shown hereafter. During the past year, this levee has been enlarged and extended, and it will doubtless confine all floods of the Arkansas except the greatest.

The Tensas Basin.—The Tensas Basin is now also completely enclosed by levees. This area extends from the southern end of the White River Basin, or Cypress Creek, to the mouth of Red River. It contains 5 370 square miles. Its drainage is through various rivers and bayous into Red River.

The Atchafalaya and Pontchartrain Basins.—Below Red River is the Atchafalaya Basin, and on the left bank below Baton Rouge is the Pontchartrain Basin, neither of which is important to the present subject.

The configuration of the great alluvial plain and of its subdivisions, and the positions of the principal stations with which this paper is concerned are shown in Fig. 1.*

Gauges.—Gauges are placed at all points where tributaries enter, and at many important intermediate stations. They are placed at variable distances apart, averaging about 40 miles. Each of the important affluents also has one or more gauges along its course. Table No. I, on page 352, gives the distances of the gauge stations on the Mississippi from the Cairo gauge, and the elevations of their respective zeros. As the height of the mean Gulf level is not accurately determined, elevations are referred to the Cairo datum, as it is called, which is a point 290.84 ft. below the zero of the Cairo gauge. The Memphis datum, which is also much used in river work, is 13.13 ft. above the Cairo datum.

* Reproduced, by permission, from Mr. Ockerson's "The Mississippi River from St. Louis to the Sea," 1892.

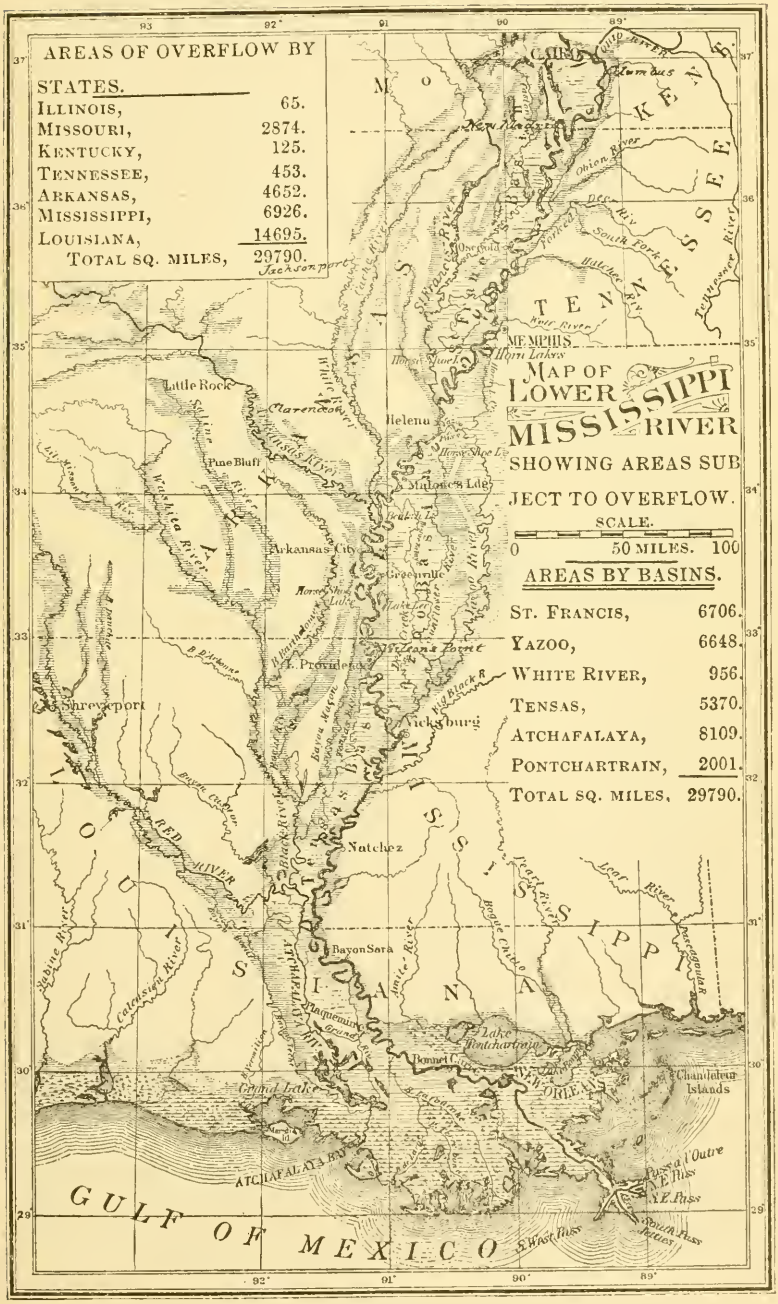

AREAS OF OVERFLOW BY STATES.
ILLINOIS, 65.
MISSOURI, 2874.
KENTUCKY, 125.
TENNESSEE, 453.
ARKANSAS, 4652.
MISSISSIPPI, 6926.
LOUISIANA, 14695.
TOTAL SQ. MILES, 29790.
MAP OF LOWER MISSISSIPPI RIVER
SHOWING AREAS SUBJECT TO OVERFLOW.
SCALE.
0 50 MILES. 100
AREAS BY BASINS.
ST. FRANCIS, 6706.
YAZOO, 6648.
WHITE RIVER, 956.
TENSAS, 5370.
ATCHAFALAYA, 8109.
PONTCHARTRAIN, 2001.
TOTAL SQ. MILES, 29790.
CAIRO
MEMPHIS
Horn Lakes
Little Rock
Helena
Pine Bluff
Arkansas City
Greenville
Vicksburg
Shreveport
Natchez
Bayon Sara
Lake Pontchartrain
Grand Lake
Chandeleur Islands
ATCHAFALAYA BAY
GULF OF MEXICO
Jacksonport
Osceola
Clarendon
Pass a l'Outre
S.E. Pass
South Pass
Jetties
S. West Pass

FIG. 1.

TABLE No. I.—Gauge Stations on the Mississippi River.

	Distance.	Elevation.
Cairo	00.0	290.84
Belmont (formerly Columbus)	21.3	287.14*
Morrison's Landing	69.0	275.80†
New Madrid	70.3	275.72
Cottonwood Point	123.0	250.62
Fulton	175.4	228.85
Memphis	230.0	203.97
Mhoon's Landing	277.3	181.48‡
Helena	306.5	161.98
Sunflower Landing	352.7	147.08
Mouth of White River	394.4	128.73§
Arkansas City	438.3	116.44‖
Greenville	478.3	108.00
Wilson's Point	531.2	93.45¶
Lake Providence	542.3	89.62
Vicksburg	599.3	66.04
Saint Joseph	648.7	52.74**
Natchez	700.3	36.89††
Red River Landing	765.1	23.85
Bayou Sara	799.8	23.95
Baton Rouge	833.3	20.06
Plaquemine	854.1	21.06
Donaldsonville	885.4	19.47‡‡
College Point	904.5	21.4
Carrollton	957.0	20.91
Fort Jackson	1 039.0	19.26§§

* This is the value formerly used. Levels of 1893 make it 287.394. The zero of the old Columbus gauge, used until June 8th, 1884, was 229.14.

† This gauge was used for New Madrid until 1882, when a new gauge was established at the latter place.

‡ The present gauge is nearly 2 miles below the former position. The change seems to have occurred at the beginning of 1892.

§ The distance is given by the Mississippi River Commission as 393.2. The gauge, however, is 1.2 miles below the mouth of the river.

‖ The elevation was given, prior to 1887, as 116.35.

¶ So for 1893. In previous years the distance is about the same, but the elevation was taken the same as at Lake Providence, namely, 89.62. This is not a regular gauge station, but is of importance from the number of discharges taken there.

** The distance is given by the Mississippi River Commission as 648.3. The gauge, however, is nearly half a mile below the town.

†† Since May, 1884, the distance of Natchez and points below has been shortened by about 12.5 miles by the cut-off at Cole's Point. This loss has been partially offset by increase of length in the bends due to caving banks, etc. The distances given in the table are the old ones, for want of accurate information as to the changes.

‡‡ Provisional.

§§ Since 1892.

The gauge at Little Rock on the Arkansas River is said to be 218 miles above Arkansas City. The elevation of the zero of the engineer's gauge is given by the Coast Survey as 241.50.

Jacksonport seems by the maps to be about 280 miles above Arkansas City on White River. The elevation of the zero of its gauge is not known. Neither is that of Clarendon on the same river. The latter place seems to be about 150 miles above Arkansas City.

Discharge Observations.—Discharges have been taken at Columbus, Point Pleasant, New Madrid, Fulton, Memphis, Hampton Landing (10 miles below Memphis), Helena, Arkansas City, Wilson's Point, Hays's Landing (10 miles below Lake Providence), Warrenton (8 miles below Vicksburg), Red River Landing and Carrollton; at Paducah, on the Ohio River; at St. Louis, on the Upper Mississippi, beside a number of isolated and scattered observations at other points. A few measurements have also been made on the lower tributaries, as at Clarendon, on the White River; at Little Rock, on the Arkansas, and at Alexandria, on Red River. A complete list, or what is intended to be such, of all discharges taken by the Mississippi River Commission or published by them is appended, and will be found in Table No. II.

TABLE No. II.—Discharge Observations Published by the Mississippi River Commission.

Place and date.	Where published.
Columbus, 1879	Report for 1894, p. 2849
Clarendon (White River), 1879	1894, p. 2855
Pine Bluff (Arkansas River), 1879	1894, p. 2852
Alexandria (Red River), 1879	1894, p. 2854
Hampton, 1879	1886, p. 2701
Fulton, 1879–80	1881, p. 98
Carrollton, 1879–80	1882, p. 112
Prescott, Wis. (Upper Mississippi), 1880–81	1882, p. 120
Winona, Minn., 1880–81	1882, p. 127
Clayton, Ia., 1880–81	1882, p. 133
Hannibal, Mo., 1880–81	1882, p. 138
Grafton, Ill., 1880–81	1882, p. 142
Saint Louis, Mo., 1880–81	1882, p. 147
Paducah (Ohio River), 1882	1883, p. 2584
Columbus, 1882	1883, p. 2607
Helena, 1882	1883, p. 2619

Place and date.	Where published.
Hays's Landing, 1882	Report for 1883, p. 2628
Red River Landing, 1882	1883, p. 2642
Bullerton, 1883	1889, p. 2649
Wilson's Point, 1883	1890, p. 3173
Carrollton, 1883, in Report for 1883, p. 2876, and	1889, p. 2642
Fulton, 1884	1889, p. 2648
Wilson's Point, 1884	1890, p. 3173
Hays's Landing, 1884	1891, p. 3488
Waterproof Cut-Off, 1884	1891, p. 3488
Point Pleasant, 1884–85	1887, p. 2818
Helena, 1884–85	1887, p. 2830
Arkansas City, 1884–85	1887, p. 2838
Warrenton, 1884–85	1887, p. 2843
Red River Landing, 1884–85	1887, p. 2852
Carrollton, 1884–85	1887, p. 2859
Wilson's Point, 1885	1890, p. 3174
Wilson's Point, 1887, in Report for 1887, p. 2888	1890, p. 3174 and 1891, p. 3496
Arkansas City, 1887, in Report for 1887, p. 2888	1890, p. 3164 and 1891, p. 3489
Wilson's Point, 1888, in Report for 1890, p. 3174, and	1891, p. 3496
Wilson's Point, 1889, in Report for 1890, p. 3174, and	1891, p. 3496
New Madrid, 1888–89, in Report for 1890, p. 3153, and	1891, p. 3501
Helena, 1888–89, in Report for 1890, p. 3160, and	1891, p. 3508
Arkansas City, 1889, in Report for 1890, p. 3167, and	1891, p. 3492
Warrenton, 1889	1891, p. 3513
Red River Landing, 1889	1891, p. 3518
Carrollton, 1889	1891, p. 3523
Below New Madrid, 1890, in Report for 1890, p. 3157, and	1891, p. 3531
Plum Point Reach, 1890	1891, p. 3534
Memphis, 1890	1891, p. 3535
Helena, 1890	Report for 1891, p. 3536
Saint Francis Basin, 1890	1891, p. 3546
Arkansas City, 1890	1891, p. 3537
Wilson's Point, 1890, in Report for 1890, p. 3175, and	1891, p. 3527
Wilson's Point, 1890 (low water)	1892, p. 3144
Warrenton, 1890	1891, p. 3538

Place and date.	Where published.
Natchez, 1890	Report for 1891, p. 3538
Red River Landing, 1890	1891, p. 3539
Baton Rouge, 1890	1891, p. 3538
Carrollton, 1890	1891, p. 3541
Cairo, 1891	1892, p. 3127
Columbus, 1891	1892, p. 3127
Plum Point Reach, 1891	1892, p. 3128
Fulton, 1891	1892, p. 3129
Memphis, 1891	1892, p. 3129
Helena, 1891	1892, p. 3130
Arkansas City, 1891	1892, p. 3132
Louisiana Bend, 1891	1892, p. 3135
Wilson's Point, 1891	1892, p. 3134
Natchez, 1891	1892, p. 3136
Red River Landing, 1891	1892, p. 3137
Carrollton, 1891	1892, p. 3138
Henry Clay Avenue (below Carrollton), 1891	1892, p. 3139
Saint Louis, 1892	1893, p. 3682
Columbus, 1892	1893, p. 3682
Fulton, 1892	1893, p. 3683
Memphis, 1892	1894, p. 2858
Helena, 1892	1893, p. 3684
Chicot City, 1892	1893, p. 3685
Arkansas City, 1892	1893, p. 3686
Wilson's Point, 1892	1893, p. 3688
Natchez, 1892	1893, p. 3690
Red River, 1892	1893, p. 3691
Carrollton, 1892	1893, p. 3693
Clarendon, 1892	1894, p. 2858
Little Rock, 1892	1893, p. 3694
Monroe (Ouachita River), 1892	1893, p. 3694
Alexandria (Red River), 1892	1893, p. 3695
Columbus, 1893	1894, p. 2825
New Madrid, 1893	1894, p. 2826
Fulton, 1893	1894, p. 2827
Helena, 1893	1894, p. 2828
Arkansas City, 1893	1894, p. 2829

Place and date.	Where published.
Wilson's Point, 1893	Report for 1894, p. 2833
Red River Landing, 1893	1894, p. 2837
Carrollton, 1893	1894, p. 2842
Little Rock, 1893	1894, p. 2840
Clarendon, 1893	1894, p. 2840

No discharges through outlets or crevasses have been included in this list.

History.

Humphreys and Abbot.—It is not now necessary to consider observations made at an early date. The discharges taken by Humphreys and Abbot were based on the assumption, supposed to be established by experiment, of an invariable datum cross-section. This having been carefully measured, the particular area corresponding to any gauge height was merely scaled off on the general plot, the area, therefore, being the same whenever the gauge showed the same reading. This assumption is now known to be erroneous. The area undergoes very great changes from day to day, most of which are accompanied by simultaneous changes of velocity. The measurement of the latter element, therefore, and not of the former, is likely to lead to very serious error.

The velocities were observed, in 1851, at as many different depths, and in as many different positions as possible. In 1858 they were taken at a uniform depth of 5 ft. below the surface. For the partial sections not adjacent to the shore, the observed velocities in 1851 were considered the mean velocities. In the shore divisions a coefficient of eight-tenths was applied for reduction.* In 1858 the observed velocities in the divisions not near the shore, at a depth of 5 ft., were reduced by means of a formula, half empirical, half rational, most laboriously worked out by a process, partly experimental, partly graphic, partly analytical, impossible to follow or to verify, consisting of combinations of curves, grouping of data and cutting and fitting generally. There results from all this hocus-pocus the following formidable expression:†

$$\frac{U_m}{U_5} = \frac{U_m}{U_m + \left(\frac{1}{3} + \frac{(0.317 + 0.06f)\,(10\,r - r^2 - 25)}{r^2}\right)(6\,v)^{\frac{1}{2}}}$$

* "Report upon the Physics and Hydraulics of the Mississippi River." By Humphreys and Abbot. Edition of 1876, pp. 234-35.

† The same, p. 276.

Coefficient of Reduction.—From this equation tables of corrections were prepared for the different observation stations, by which the observed velocities were finally reduced. The upshot of it all is that the ratio between the mean of all velocities in vertical planes, at a depth of 5 ft., and the mean of all the mean velocities in vertical planes, in calm weather, varies from 0.97 for low velocities to 0.99 for high. This result does not agree at all with later determinations. According to the elaborate measurements of 1882, the ratio, at all stages and stations, was about 0.91. The data supplied by Humphreys and Abbot, moreover, are very scanty, and are of no use for exact investigation. The results announced by them can be accepted, therefore, only as approximations.

The errors which appear in the work of these pioneers of experimental river hydraulics should not detract from their merits. When the entire novelty of the problem set before them is considered, the want of any precedents to guide them, the magnitude of the task, the imperfect means at their command, the short time allowed for the work and the rapidity with which it was performed and the results matured and announced, their achievements seem little short of herculean. That they made mistakes is not surprising. There was as much of the soldier as of the engineer in General Humphreys, and his combative and positive character has given a deep tinge to all his opinions. Nevertheless, the work must always stand as a monument of original research, of colossal labor and of engineering genius. All the work done since has been based on the model set by them. Engineers of the present day owe them an everlasting debt of gratitude. Used with discrimination, their work is still of great value.

Low-Water Board of 1879.—The present system of gauges was organized in 1871, and the records extend back to the latter part of that year.

In 1879, a number of discharges were taken on the Mississippi and some of its tributaries by Colonel Suter and Colonel Benyaurd. The results of these have only lately been made public. They have frequently been made use of by Colonel Suter and others as the basis for calculations. They are generally known as the work of the "Low-Water Board" of 1879.

The Mississippi River Commission, 1879-80.—In 1879 the Mississippi River Commission was organized; and in the fall of that year, under

the direction of the secretary, Captain Leach, discharges were taken at Fulton and at Carrollton, and observations for slope, cross-section, etc., were made at Lake Providence. These were by far the most comprehensive and accurate series of gaugings yet made. They occupied a whole year, and covered a very wide field, embracing the determination of curves of vertical velocities, transportation of sediment, measurement and travel of sand waves, variations of slope and cross-section, etc.

Current and Sediment Observations of 1880-81.—In 1880-81 a series of observations was taken on the upper Mississippi from Prescott, Wis., to St. Louis, now most useful for its careful data on the subject of the transportation of sediment in suspension.

Observations of 1882.—These works were a preparation for the yet more important labors of 1882. In this year occurred the greatest flood of which there is any authentic or accurate record. Parties were early placed in the field. Discharges were taken at Paducah, on the Ohio River, just above Cairo ; at Columbus, Helena, Hays's Landing and Red River Landing on the Mississippi. At the same time, measures were taken for the determination of the escape over the banks into the Saint Francis Basin, and the unprotected parts of the White River, Yazoo and Tensas Basins, as well as through the numerous breaks in the levees of all the basins which occurred during this portentous flood. A mass of information was thus gathered which, imperfect as it is, furnishes most of the existing knowledge of the maximum flow of the Mississippi, and has contributed greatly to the correct understanding of the general movements of that mighty stream.

The necessity for great accuracy in the measurements of the discharge was not at first apparent. Had it been never so evident, the means were not at hand for its attainment. The men were comparatively untrained, the methods new, the outfits wholly inadequate, scraped together in a hurry, and eked out often with makeshifts and home-made devices, adapted to the work by the genius or non-genius of the engineer in charge. The current meter was at that time a comparatively untried instrument, and the form in use was not suited to the turbulent, muddy, drift-laden Mississippi. Existing formulas for the use of floats were often inapplicable to the new conditions. New sources of error were discovered. Base lines were wrongly

measured in the hurry. Lead-lines had stretched several feet before the error was detected. Instruments were broken by rough usage and could not be replaced, or sometimes failed to do the work expected of them. Notwithstanding all these drawbacks, the measurements are not extremely discordant with one another or with the probabilities of the case, and indeed sometimes exhibit fewer discrepancies than others taken with more precautions and refinements. The records, as they are presented, appear in their native nakedness. No attempts have been made to reconcile their discrepancies. Indeed, they greatly need an intelligent discussion, such as has been bestowed on those of 1884-85. The notes which accompany the reports are usually very meagre, and give very little insight into special sources of error which were undoubtedly present.

In 1883 and 1884 measurements were continued of the escapes of water through crevasses and outlets and over banks.

Observations of 1884–85.—The next noteworthy series of observations was taken in 1884–85, extending from about October to April. Complete measurements were taken at Point Pleasant, Helena, Arkansas City, Hays's Landing, Warrenton (below Vicksburg), Red River Landing and Carrollton. These observations were published in 1887, with a full discussion and very satisfactory notes by L. L. Wheeler, M. Am. Soc. C. E. Mr. Wheeler's comments throw much light on the difficulties which attend the taking and the reduction of discharge observations, and point out the means of correcting many errors.

Observations of 1889.—From the failure of appropriations very little was done in the way of gauging the flow of the river until 1889, when a series of measurements covering four or five months was made at New Madrid, Helena, Arkansas City, Warrenton, Red River Landing and Carrollton. Full details are given of methods, etc.; 1889 was a year of low water, and the observations embrace only medium stages.

High-Water Discharges of 1890, 1891, 1892 and 1893.—The years 1890, 1891, 1892 and 1893 were all high-water years, and discharges were taken at several of the stations during the period of flood, and sometimes isolated measurements were made at low water. Many of the observations were taken with extreme care, and in almost all cases the data are accompanied by complete explanatory notes. Sometimes very curious phenomena are revealed by these high-water observa-

tions, and some of the apparent anomalies which they present have not yet been satisfactorily explained.

Measurement of Cross-Sections.

It is not intended to give an essay on the practice of gauging rivers, which would of itself require an elaborate paper, and would demand for its treatment much experience on the part of the author. It is only desired to call attention to the sources of the many errors and discrepancies in the records of this sort of work, and especially in the case of the Mississippi, with the view of discriminating between real errors and mere apparent anomalies or puzzling incidents, which may be very genuine and may afford the key to the discovery of new laws.

Soundings.—Depths have been measured from the earliest times in very nearly the same way. In Humphreys and Abbot's day, in 1879–80, and generally up to a recent date, soundings have usually been taken by the "drifting" method, the line being lowered to a position near the bottom just before passing the range line, and dropped to actual contact at the moment of crossing. From a skiff, catamaran or launch, doubtless this is the best way. Where a steamboat is used, it is customary, at the present time, to proceed as in taking deep-sea soundings on shipboard, that is, the line is stretched fore and aft, along the guards of the boat, the lead being in front. The lead is then heaved, and the sounding is read as the line comes to a perpendicular at the after part of the boat. Either method is good enough if due pains are used to insure accuracy.

The first, most obvious and most necessary precaution to be taken is to see that the lead-line is of the correct length. In the careful observations of the present day, the line is tested before and after each day's work and the necessary correction inserted in the field-book opposite each sounding. The notes accompanying the earlier observations give very few particulars in this regard. Humphreys and Abbot used a welded chain. The common lead-line has been employed by almost all others, however, often checked by the occasional use of piano wire. Almost no assurances are given in the work of 1882 that any pains were taken to insure accuracy of measurement in depths. In Mr. Wheeler's discussion of the observations of 1884–85 there are many instances showing the possibilities of error that exist in taking

soundings, and likewise the frequency of such errors. At Helena, on October 11th, the lead-line was tested and was found to have shrunk 3 ft. in 50 ft., and there were no data for determining what the error was on previous days. At Arkansas City the corrections were made once a week, "but on some occasions large changes took place in the lead-line with no means of determining just when the change occurred." At Warrenton the soundings were corrected for errors of the lead-line. The corrections during the latter part of the season were very large, being about 12 ft. in 70 ft. At Red River Landing no tests of the lead-line are recorded in the notes, and no corrections are given or have been applied. Mr. Wheeler notes that a large change took place in the datum area between January 3d and January 6th. "It is much to be doubted if such a change really took place. The only note which has any relation to the subject is a statement in the journal that a new lead-line was tagged January 5th. It is also probable that a heavier lead came into use on January 6th." The change thus recorded was about 10 000 sq. ft. of area, corresponding to 2.4 ft. of mean depth.

The location of the sounding stations is also of great importance. If the exact position of the point where each sounding is taken is uncertain, how can one day's observations be compared with another's? The usual practice has been, first to establish a range line by proper signals or targets, and then to locate velocity stations by the intersection of oblique lines, each defined by two numbered targets, on one or the other bank. Soundings are often taken at the velocity stations, but not necessarily; but as the measurements of depth are taken at much shorter intervals than the measurements of velocity (50 ft. as compared with 200 ft. or more), the intermediate stations have been fixed by angles taken from a base line by a transit on shore or a sextant from the boat. To show the possibilities of error in this particular, reference is again made to Mr. Wheeler's discussion. At Point Pleasant soundings were located both by signals and by angles.

"The locations by angles and by signals in the main are quite discrepant, and it has been found impossible to reconcile the two methods of location. The notes of the location have not been recorded or the computations of the same preserved, so that it is now impossible to prove that the signals were in the right places to make intersections 200 ft. apart. On the other hand, the sextant angles show discrepancies among themselves, so that it is certain that the sextant locations are erroneous. There is also some evidence that the signal locations

are erroneous, but as all velocity measurements and all soundings after January 1st, 1885, were located by these signals alone, it was decided to reject the sextant angles entirely and depend upon the signals for location."

In the early part of the work some very large changes took place in datum areas, and in such a manner as to place in doubt the measurements of areas at that time.

At Warrenton, positions were located with a pocket sextant, too small an instrument to give accurate locations with the small angle employed. The soundings were not taken at regular intervals, and up to December 15th, the velocities were also measured at irregular distances.

At Red River Landing, the locations of the soundings were determined both by signals and by sextant angles. The two locations do not agree exactly, but the difference is small.

At Carrollton, the soundings were taken at irregular intervals, and were located by the series of signals on the right bank. In the revision many differences were encountered. Some of them could be settled by a reference to the original notes, but such as could not were settled arbitrarily. In some few instances the notes have been corrected arbitrarily where they were evidently largely in error. "On a large number of days velocities were observed, but no soundings were taken. There seems to have been no good reason for this. It was decided to interpolate depths for those days when soundings were not taken. In this manner depths have been interpolated for 39 dates."

Degree of Accuracy Attainable.—As a measure of the accuracy attainable in these locations, at Wilson's Point, in 1891, soundings were located simultaneously by a transit on shore with a base of 1 600 ft. on the Louisiana shore, and by a sextant on the boat to a base of 1 000 ft. on each bank. The difference varied from 0 to 39 ft., the latter, however, being exceptional. The discrepancy generally did not exceed 6 ft.; and the mean difference of 94 sets, including the extreme case, was 2.3 ft.

It is the opinion of the engineers in personal charge of this work that it is possible to measure the cross-section with a near approach to accuracy; and that the figures given in the reports of discharge observations as representing the several areas are, in the main, nearly correct.

It is to be remembered, however, that a change in method or a change in the personal constitution of the party may very easily cause an apparent change in area. For instance, the substitution of a new leadsman may make an immediate apparent scour or fill where none really existed. The change of a single foot in a series of soundings will make a change of 4 000 sq. ft. or so of area; and a change of a foot in depths of 60 to 100 ft. is easily made.

Changes of Cross-Section.—Frequently very large changes of cross-section occur from day to day, being generally afterwards compensated by a change or changes, sudden or gradual, in the opposite direction. It is the opinion of the observers that these changes are usually genuine. They may be due to the passage of sand waves, to violent caving or filling, or to various causes, some of which are yet unknown or obscure. In corroboration of the position that these changes are real, it is to be noted that the violent fluctuations in question are usually accompanied by as sudden a change, in the inverse sense, of velocity; which would not be the case if the discrepancies were fictitious and proceeded from errors of measurement—unless by an occasional coincidence.

Measurement of Velocities.

The current meter is used almost to the exclusion of all other means on the Mississippi, at the present time, other methods being employed only occasionally as checks or under exceptional circumstances. Many observations of importance, however, have been taken by floats; and in view of certain objections which experience has shown to be attached to the current meter, engineers are sometimes found who are disposed to recur to former methods.

Floats.—The systems of measurements of velocity by double floats and by rod floats are well understood. In the former, there is no essential difference at the present day from the method pursued by Humphreys and Abbot. In the latter, there are some modifications in the Mississippi practice, owing to the great depths existing. Rod floats cannot be successfully handled if more than 35 or 40 ft. long.* Consequently, in very deep water, such as is habitually found throughout the greater part of the cross-section in the Mississippi in time of flood, the float does not reach half way to the bottom.

* Mr. J. H. Davis, in "Report of the Mississippi River Commission for 1883," pp. 2601, 2616.

It does not, therefore, fulfil its mission of determining the mean velocity in each vertical by direct observation, but the results have to be reduced by a formula. It has, to be sure, some advantages over a surface float or one immersed only 5 ft., because it is influenced by a greater number of filaments, and thus many accidental and temporary irregularities are eliminated.

As to the confidence to be placed in the results furnished by these instruments, the testimony of the engineers who have made actual use of them in work on the Mississippi may be taken. Says Captain Leach, at Carrollton, in 1879–80:*

"In a series of double floats run in rapid succession, at the same depth and over the same path, the individual results have been observed to vary by as much as 25 per cent. The velocity of a single float may, by accident, be the average velocity for that portion of the stream, but in general it will not; and however exactly it may be measured, it cannot be accepted as more than an approximation to the quantity desired. Add to this the error in determining the cross-section, and the uncertainty as to whether the section measured is the really effective one, and it seems improbable that these discharges can be relied on within 10 per cent."

At Paducah, in 1882, W. G. Price, M. Am. Soc. C. E., says:†

"I think double floats do not give the mid-depth velocity, owing to the effect the current has on the upper float and connecting wire. The upper float is also affected by the wind. For this reason the measurement would be too great except where there is a strong up-stream wind, when it might be too small. Measurements of vertical curves of velocity with double floats in an up-stream wind show a much greater decrease of velocity near the surface than the same measurements with the current meter.

"The mid-depth velocity is not always a mean velocity. I think it varies with different sections and with different stages of water, being from 1 to 4 per cent.

"There is no check on any of the observers, one of whom being a careless man will spoil the accuracy of the work. The shore observers may work with their instruments out of adjustment; they may miss the float, but give the signal as though all was right; or the floats, to save labor, may not be adjusted to mid-depth, and the chief of the party may not know it.

"With rod floats, when they are observed from the shore, as with double floats, there is no check on any of the observers. Careless observers will let the float project too much out of water, so that it

* "Report of the Mississippi River Commission for 1882," p. 101.

† "Report of the Mississippi River Commission for 1883," p. 2584.

is affected by the wind. In deep water floats often do not reach to mid-depth.

"As the vertical curve of velocity varies very much with different sections, and also at different points on the same section, it is impossible to make a formula which will reduce the observed velocity to a true mean. For this reason, when the floats do not reach nearly to the bottom, they may not be as accurate as the double floats.

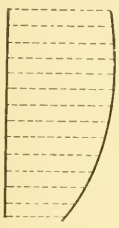


FIG. 2.

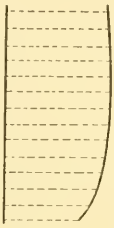


FIG. 3.

"During the time these measurements were taken the river fell from about a medium stage to low water. At first the rod float discharges were the greater, but when the river had fallen much lower, they were less than those taken with the meter. When the rod float discharges were the greater, the vertical curves of velocity were like Fig. 2, and when they were less, they were like Fig. 3. This showed that the observed velocity of the floats had not been reduced enough at first, and afterwards too much, though the same formula (Francis's) was used all the time."

Mr. J. H. Davis, at Columbus, in 1882, remarks:*

"I was led to believe that at times the surface and subsurface parts were acted upon by almost opposite forces. At times the surface float would be suddenly drawn to right or left, in which case it would be considerably inclined and partially immersed. Sometimes it would incline backward or forward. In each of these cases the surface float had a powerful influence in retarding or accelerating the sub-float; the velocity of the former not representing the correct velocity of the water in the vicinity of the latter. The above indications were frequently noticeable in high water. Sometimes also in high water the sub-float was apparently held up, the assumed immersion not being reached. This was shown by the fact that several feet of wire were sometimes taken in before there was any appreciable weight of the sub-float. By an examination of the vertical velocity observations at Fulton, it will be found that in the sets in which two or more floats were run at the same depth the velocities differ from 1 to 15 per cent."

Current Meters.—With current meters, very much depends on the kind of instrument employed, its suitability to the particular class of work it is called upon to do, its mechanical excellence, the care exercised in keeping it properly adjusted, its manipulation, its rating and the accuracy with which its indications are recorded and the results

* "Report of the Mississippi River Commission for 1883," p. 2616.

computed. It is a delicate instrument, with several bearings and an electrical connection, and its ratings are found to vary considerably with even slight changes in repair or maintenance.

The first forms of current meter which were devised were of the windmill, screw or propeller pattern; that is, the vanes which were moved by the current revolved in a plane perpendicular to the direction of the latter. Such was the design of the original Woltmann meter, of the Révy meter which was used on the Paraná, Uruguay and La Plata Rivers in 1871, of the Harlacher meter and of several others. The instruments used in the Mississippi work are constructed on a different principle. They are of the "anemometer" type, as it is sometimes called; that is, the vanes are replaced by cups on the ends of arms which revolve in a plane parallel to the thread of the current. The motive force here is the difference between the pressure exerted by moving water on a concave and on a convex surface. The meters used in the early experimental work on the Mississippi were of the type devised by General Ellis for use on the Connecticut River in 1877, and were of the anemometer pattern. The changes which the experience of 1882 and of subsequent years showed to be desirable in the Ellis meter have resulted in the evolution of the Price meter, which is now in general use. Of late, there has been a disposition to revise the decision made in favor of the anemometer type of meter, and to return to the screw plan, as in the new Ritchie and Haskell meter; so the question cannot be considered settled.

There are at least two sources of error in the use of any kind of meter. First, the instruments or the observers may not tell the truth. Second, they may record and transmit with perfect accuracy what has happened; yet this récital of facts may not be at all what is sought, although apparently so. The meter may deliver with absolute truthfulness its tale of so many revolutions in so many minutes, yet in attempting to translate these into feet per second grievous blunders may be made, more especially when it is assumed that this travel of a thread of water was in a direction parallel to the banks of the river and to the surface of the stream; and still more particularly when it is accepted as a fact that this solitary observation represents the mean velocity of a large portion of a swift and turbulent river; the interval between the observation stations being never less than 150 ft. and frequently as much as 300 ft., and the depth often from 50 to 100 ft.

A great part of the knowledge of the flow of the river rests upon the testimony of these mute and unintelligent witnesses. It is believed that they tell their story with nearly perfect fidelity, as far as they go. There is a great deal that they do not tell. It is the engineer's part to interpret their deliverances correctly, to divine or to ascertain from other sources the information which the meters fail to give, and generally to supply their deficiencies.

The first point to be considered is the mechanical perfection of the instrument. Old and worn meters will not give correct indications; they will register the number of revolutions probably, but this number will not correspond to a definite velocity in feet.

Secondly, the meter must be maintained in perfect order, the bearings and connections kept in a uniform state as far as possible, and, every time a change of any kind is made, the instrument should be rerated. It is the experience of the observers that when a meter is taken apart and cleaned, its rate changes; that when a new electrical connection is put in, a bearing oiled, or, in short, the slightest alteration made, the effect is immediately perceptible and is often considerable.

Rating of Meters.—Assuming the instrument to be in perfect condition, it is essential that it should be correctly rated. It is thought by many observers that here lies the *nodus* of the difficulty. The operation is simple enough in principle. It consists merely in dragging the meter through still water at a rate determined by observation. The method now approved is to lower the instrument from a skiff to a distance of 5 ft. or more in a pool of considerable depth, and then have the skiff towed by a rope between fixed range lines, taking the time by a stop watch. Mr. Charles H. Miller, an observer of ability and experience, declares * that the skiff should never be pulled with oars, and that the observers should be cautioned about keeping the same positions and not swinging their bodies about during the observation. "In a rating taken from a steamer drawing 2.5 to 3 ft. of water, with the meter in 10 ft. of water, a large error was found over other ratings of the same meter, and it was then rated at 20 ft. under the surface." Mr. Miller thinks a slight current or even an oscillation not objectionable if reasonably constant, care being taken to get the observations in pairs, and to have the times of transit the same both ways.

* See *Engineering News*, January 24th, 1895.

Ratings in shallow water are not to be relied on, nor where the meter is brought too near to the skiff or steamboat.

These matters were not understood at first, and it is only by patient observation and practice that the importance of extreme care in rating has become known. It is thought by some skilled engineers that many of the discrepancies hitherto unaccounted for are due to careless or unskilful rating.

A meter wheel revolves under the impulse of moving water, and against the resistance of friction. The object of rating is to ascertain the ratio between a revolution of the wheel and the velocity of the current. If the wheel makes one revolution in a second, while the water moves 2 ft., then the ratio is 2 to 1. If there were no friction this ratio would be constant, and three revolutions would correspond to a velocity of 6 ft. The same result would follow if the resistance of friction were proportional to the velocity, and the equation for rating the meter would be $y = a\,x$; where y is the velocity sought, x the number of revolutions in a second, and a a constant expressing the relation between them. The friction of solids, however, is supposed to be nearly independent of the velocity. The resistance of the bearings will therefore be nearly the same at low velocities as at high, and there will be a greater proportionate resistance in the former case. Therefore, there will be fewer revolutions of the wheel in proportion to the velocity. One revolution of the wheel will no longer represent 2 ft. of velocity, but, perhaps, 3 ft. Therefore, an expression is desired which shall show this varying relation. General Ellis * gave as this equation the formula $y = a\,x + b$, in which b is the term expressive of the constant resistance of friction. The same equation was adopted by Mr. Price and was used in the reductions of 1882 and of several subsequent years; in fact, until very recently.

There are several elements, however, which enter implicitly into this expression and influence the values of a and b and prevent them from being constants. The quantity a, so far as it depends upon the theoretical rate of travel of the perimeter of the wheel, cannot be regarded as constant, for there is the "slip" to be considered, the law of which is unknown, and the unequal action of the filaments upon the arms of the wheel at different angles, accelerating in one direction and retarding in another. The term b, so far as it depends upon the fric-

* See "Report of the Chief of Engineers for 1878," Vol. II., p. 308.

tion of the bearings, cannot be constant, for it will be influenced by the perfection of the machinery, the proper lubrication of the same, etc., all of which have been shown to be subject to serious irregularities.

The friction of the bearings and working parts is not the only resistance to be considered, for the action of the water against the convex surface of the cups or vanes, which are moving contrary to the current, is also important. The motion of the meter, indeed, depends upon the difference between the forces exerted by moving water upon cup-shaped and wedge-shaped surfaces. These forces are governed by laws altogether different from those of solid friction, for they depend on the square of the velocity. How important a part is

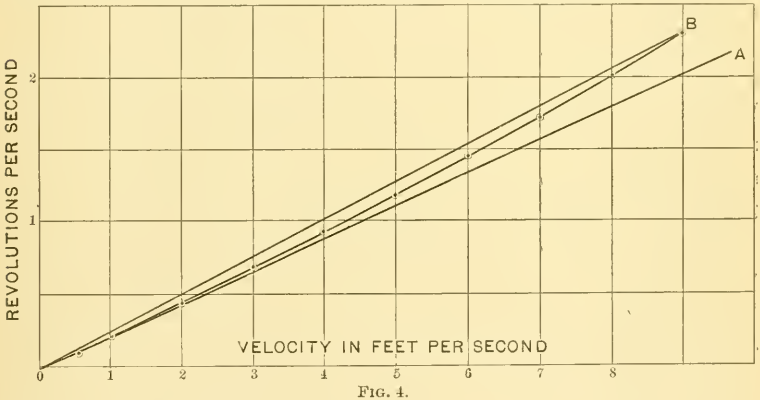


FIG. 4.

played by the total resistance is apparent from the fact that the meter will not register very low velocities at all.

These reasons seem to be sufficient to explain the fact that the relation between the revolutions of the meter and the velocity is not at all a simple one, and that the formula expressing that relation is not the equation of a straight line. It approximates to it, upon a casual inspection, so that the line, when plotted from actual experiment, upon a large scale, is not easily distinguishable from a straight line by the eye. It is not surprising, then, that the earlier observations were reduced upon that supposition. The constants were ascertained by combining a number of equations of condition, derived from observation, on the principle of least squares.

The error introduced by this assumption is very material. In a given instance, taken from actual practice,* if a tangent, OA, be

*At Arkansas City, May 18th-23d, 1893. The observations were taken with all possible care, with a new and well tested meter, by Mr. Thomas C. J. Baily, Jr., U. S. Surveyor, and with every precaution suggested by previous difficulties.

drawn to the lower part of the curve *O B*, it will diverge from the curve (see Fig. 4) at 6 ft. by 0.5 ft., at 7 ft. by nearly 0.7 ft., at 8 ft. by more than 0.9 ft. If a chord, *O B*, be drawn from 0 to 9 ft. it will depart in the middle of the curve by about 0.4 ft. If a straight line be drawn so as to coincide, as nearly as may be, with the curve, it will depart from the curve at several points from 0.1 to 0.2 ft., sometimes in excess, sometimes in deficiency. The low velocities will be too high, the medium velocities too low, the high velocities too high, and the percentage of error will be very different for the several cases. In the low velocities it may be 10 or 20%; in the high, only 1 or 2 per cent. It is obvious that by such a method, while the actual error at each point might not be very great, yet it might be sufficient to obscure all traces of the law governing the progression of the discharge, intricate as this is, and influenced by numerous delicate perturbations. When the circumstances of the observations, the accuracy of the instrument, its condition, the skill, experience, accuracy, judgment and good faith of the observers are unknown, it is not easy to assign the limits of probable error in rating meters.

Present Practice in Rating Meters.—It is the rule at present to rate meters frequently, and to apply the ratings directly to the observations taken during the adjacent period. This has only lately been the practice. In the older observations, notably in 1884–85, meters were rated only occasionally, perhaps once in a season, and apparently with little care or method. In 1882, so far as the record shows, the meters were rated frequently, and there is no reason to suppose that the methods employed were less accurate than those of 1884–85. Indeed, in some respects, they were perhaps better. The meters, however, were mostly too light and delicate for the work, the methods were novel, and most of the observers without experience.

Nowadays, from the rating observations a line is plotted, the revolutions being the ordinates, and the velocities the abscissas. Fig. 4, just referred to, is an example of this work. From the line thus obtained, the number of revolutions being given, the velocity may be taken by inspection. In short, every precaution is taken that the instruments and the men shall tell the truth. Whatever errors occur are mostly due to difficulties or faults in the method. The difficulties are mostly of two classes—difficulties of manipulation, or execution, and difficulties of situation. Supposing these to be overcome, there are still faults in the method.

Difficulties of Manipulation.—In a stream like the Mississippi, and especially at high water, it is almost or quite impracticable to take discharges from an anchored boat, and, indeed, it is doubtful whether an anchored boat could be kept much more quiet than a boat under steam; therefore the boat is always kept under way. The object is to hold it on the range line, and as nearly as possible at the velocity station, while the observation is in progress. With all possible skill in management, there will be a little drift, direct and lateral. Care is always taken, however, to see that the boat is on the range line at the beginning and the end of the observation, so that the excursions up and down stream may compensate one another. Lack of this precaution has often caused serious errors. It is the belief of good observers that lateral drift may be kept within narrow bounds, and that the error should be inconsiderable.

It is not always practicable in swift currents and at great depths to hold the meter in a horizontal position. If it is tilted at an angle, the bearings come at places where it was not designed that they should, and in old instruments especially the rate is greatly influenced thereby.

Winds and waves do not influence the mean velocity, but they shift the axis of the curve of vertical velocities, and instead of being at nearly 0.6 of the depth, as usual, the place of mean velocity in the vertical may be at 0.1 or at 0.8. Again, the waves produce a rocking or an up-and-down motion of the boat that is communicated to the meter, and the latter registers these movements as well as the velocity of the water. Large perturbations of the discharge have been produced in this way.

Difficulties of Situation.—With all the precautions that experience has suggested, there are sometimes discrepancies or anomalies in discharge observations so great that they cannot be accounted for in any of the ways heretofore indicated. Instructive examples of this are to be found in the observations taken in 1891 and 1892 at Arkansas City and at Wilson's Point. Up to this time it had been impossible to check the observations at one point by those taken at another, owing to the intervention of tributaries, which introduced a disturbing element difficult to calculate. Thus Helena could not be checked by Arkansas City in 1884–85, because the White and Arkansas Rivers poured out between the two stations a variable and unmeasured volume, which sometimes amounted to 300 000 cu. ft. per second. In 1891,

however, C. McD. Townsend, M. Am. Soc. C. E., being dissatisfied like everybody else with the existing state of the knowledge of discharges, had simultaneous gaugings taken at the two places above named, 93 miles apart, and without any tributary or other disturbing condition between. The results showed the discharge at Arkansas City to be 150 000 to 200 000 ft. more than at Wilson's Point. Had the difference been only for a day or two it might have been explained, well or ill; but it was continuous and persistent. At Arkansas City, from March 28th to April 4th, the records were 1 390 586, 1 419 422, 1 423 730, 1 408 553, 1 419 321, 1 405 641. At Wilson's Point they were 1 272 440, 1 288 000, 1 219 436, 1 200 578, 1 247 445, 1 275 904, 1 234 182. Averages, 1 411 209 and 1 248 283. There was no crevasse or other disturbing influence between them. The same discrepancy kept up till April 23d, after which date the difference slowly diminished. This unexpected result put the engineers to their wits' end. The meters were compared and exchanged. They had been several years in service, and their bearings were somewhat worn. They were rated frequently, and the rates of the same meter at different times were found to be very discrepant.

Captain Townsend considered the meters to be unreliable, and ordered two new ones. The next spring, 1892, a still greater flood developed, and high-water discharges were taken at the same two places with the new instruments. The results were more discrepant than before. The discharges at Arkansas City from April 26th to May 9th averaged 1 598 682; at Wilson's Point, 1 364 277. The instruments were compared, side by side, and meters and observers were changed, without affecting the result. There were no disturbing influences between the two places except the overflow of the foreshores outside of the levees, which increased only 1.7 ft. during the interval of 13 days.

Several natural explanations offered themselves, but none were adequate to explain so great a discrepancy. The reservoir space of the whole area included between levees from Arkansas City to Wilson's Point is only about 228 miles. An excess of discharge at the upper station of 200 000 cu. ft. per second would fill the whole reservoir 2.7 ft. deep in one day. The actual increase of depth was only 0.1 or 0.2 ft. The loss of such a volume by evaporation or percolation was out of the question. Besides, there was no general loss of volume of discharge coming down stream, as shown at other stations.

Obliquity of Current.—A plausible solution of the difficulty was suggested, it is believed, by General Comstock, and Captain Townsend is disposed to accept it. This attributes the excessive registration at Arkansas City to obliquity of current. It is known that in bends there is a lateral circulation of currents from bank to bank,* the upper current being centrifugal, the lower centripetal. So in "crossings," at the reversion points of bends, the current leaves the one bank to occupy the other. The level of the water on the left bank is sometimes as much as 1 ft. above the level of the water at a point directly opposite on the right, or the reverse. This difference of head produces a strong lateral movement, which, acting in conjunction with the regular down-stream flow, has a resultant acting at a considerable angle with the axis of the stream. Captain Townsend had float observations made in 1891 for determining the direction of the current. At Arkansas City the average deflection was about 10° 30′, indicating a discharge, as observed with meters, about 1.5% too great. At Wilson's Point, also, subsurface floats were run for the same purpose. The result showed a deviation

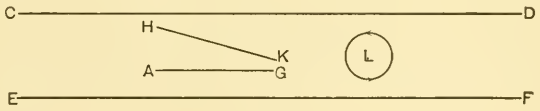


FIG. 5.

even greater than at Arkansas City, namely, about 12°, indicating an excess in measurement of about 2 per cent.

Vertical Currents.—It is likely that vertical currents also exert a powerful effect. What is demanded of the meter is that it shall give the velocity of a filament in each vertical which runs parallel to the general direction of the bank and to the bottom, or, rather, surface. What the meter gives is the velocity of the filament in the direction in which the latter is moving at the time, be that direction direct, oblique, horizontal, inclined or vertical. If Fig. 5 be taken to represent a river in plan, CD and EF being the banks, then what is sought is the velocity in the direction GA. What the meter gives is perhaps the velocity in the direction KH. If the sketch, however, be taken to represent the profile of the river, of which CD is the water surface and EF the bottom, then the velocity is sought in the direction GA in a horizontal plane. It is obtained perhaps in the inclined

* See "Encyclopædia Britannica," vol. xii, pp. 498–9, article, "Hydromechanics."

direction KH. If the inclination was known in each case, the projection of KH on GA, which would be the quantity sought, could be obtained easily. Generally the inclination is unknown. In time of flood, when discharge observations are most important and are generally taken, a depth of 0.6 implies a submersion of 40 ft. or more, where the movements of the meter cannot be observed, and can be only partially controlled.

The general direction of the current at the surface or at any desired depth may be ascertained with more or less accuracy by the use of floats. This is a step in the right direction. But what is wanted is the particular position of the meter, both in horizontal and vertical planes, during the time of observation at each station. A considerable uncertainty must always be felt as to the indications of ordinary meters, especially at the period of high water, even though the general obliquity of the current be known, from the great irregularity and turbulency of the current at such times. Should the meter, at the time of registration, encounter an eddy, in a horizontal plane, or a "boil" in a vertical plane, as at L in Fig. 5, the indicated velocity and direction may be valueless, unless both velocity and direction be taken at the same moment of time. Several ingenious devices have been proposed for this purpose, and others are under projection. The "direction current meter" of Messrs. Ritchie and Haskell* is an instrument by which the lateral deviation of the meter is ascertained by means of a submerged compass, the travel of the needle of which is recorded on a dial above water by an electrical connection. It is not known what measure of success has been attained.

Other Disturbing Influences.—There are a number of disturbing influences, which, like the above, may be classified under the head of "difficulties of situation," which affect the velocity at any station from time to time, and render any registration which occupies only a few moments liable to error of greater or less amount. Some of them have been referred to. All are imperfectly known, when known at all, and doubtless there are others which are yet unsuspected. A competent and experienced observer,† stationed at Arkansas City in 1893, remarks:

---

* Described in *Engineering News*, January 10th, 1895.

† Mr. T. C. J. Baily, Jr., in "Report of the Mississippi River Commission for 1894," p. 2815.

"When the river is in a normal condition, the discharge varies but slightly within a short time; but when near its limits, immense fluctuations occur within small intervals. This will be better appreciated by an inspection of the velocity curves. These show that when the river is very high, the immense flats above will flow out and fill in within a very few minutes of each other, affecting the flow by at least 100 000 cu. ft. per second."

The same engineer, when stationed at Wilson's Point, in 1892, reports:*

"Two thousand feet above the range on the Mississippi side, the Duncansby Chute pours an immense volume of water into the river at a high velocity, the axis of the chute forming an angle of 39° with that of the river at the point of entrance. It is believed that this water, by eating out the bar which has formerly existed at the east end of the section, is one of the causes of the great variations which may be observed from day to day in the discharges. Stations 2, 3 and 4 were the ones affected, the current varying on them so greatly from minute to minute that it was extremely difficult to judge when a true average velocity had been obtained. For this reason, these stations were occupied from 8 to 20 minutes. * * * From June 6th until the end, the current on Station 1 was running up stream as determined by double floats, and the entire partial discharge of this section thereafter was subtracted from the sum of the remaining partial discharges. The position of the seam between the up and down stream currents was not at all constant, it shifting from a point 50 ft. outside of Station 1 to Station 2 within a minute."

Discharges taken at Chicot City, a few miles above Arkansas City, in 1892, showed differences of 100 000 ft. from those taken at the latter point, and this sometimes in excess, sometimes in deficiency.

It is thus seen that, to obtain correct discharges, or discharges approximating correctness, especially at high water, much depends on the skill, experience, conscientiousness and judgment of the observer. Even with the utmost skill and care, a discharge observation made in the ordinary way is simply the determination of the velocities prevailing at the moment at 10 or 15 or at most 20 isolated points distributed throughout a cross-section of 200 000 sq. ft. or more, together with the sounding of the section at intervals of 50 ft. or so. That results obtained in this way should not accurately represent the true discharge of the river is not surprising. If they did represent the true discharge for the time being, it might be very far from the mean discharge for the whole day.

* "Report of the Mississippi River Commission for 1893," p. 3669.

The "Flanking Method."—To eliminate, as far as possible, the effect of lateral obliquity of current, and to obtain a more nearly true mean of the velocity of the numerous fillets of water from the one side of the river to the other, Captain Townsend has tried a method which he calls the "flanking method," which consists in starting from one side of the river with the meter submerged to a certain depth, usually 20 ft., and slowly "flanking," or moving across, keeping the steamboat's head up stream, and keeping on the discharge range, having the record running continuously, but noting the registrations and the time as each intersecting range is passed. This operation is repeated at once in the reverse direction, the two trips constituting one complete observation.* Of course the results are corrected for the sidewise motion of the boat.

This method was tried, in conjunction with the ordinary plan, in 1893, at both Arkansas City and Wilson's Point, and was found to afford a valuable check. The results of the two trips from bank to bank usually show the existence of a lateral current, though not a very strong one, at the depth of immersion of the meter, the difference of velocities going and coming seldom exceeding 0.2 ft. and generally being less than one-tenth. They are more uniform than the results obtained by the old plan, and have occasionally corrected the excessive figures of the latter. One day, for instance, at Arkansas City, the discharge found by the old or stationary method was 1 667 306; by the flanking method, 1 566 054. On the whole, the two tally more closely than might have been expected.

In 1893, the discharges at Arkansas City and Wilson's Point agree fairly well; and when checked by the results of the flanking observations, they present a very coherent mass of data, affording the most trustworthy measurement of the extreme high-water discharge ever obtained on the Mississippi. They show, among other things, that there is no considerable loss of volume between the two stations by evaporation or leakage, and that between two stations so closely adjoining, with a nearly stationary river, there is but little weight to be given to the reservoir effect of the channel and the overflowed bank.

Possible Error in Deficiency.—The discharges of 1891 point to the possibility of an error in deficiency. From the probabilities of the

* Report of Mr. Charles H. Miller on observations at Wilson's Point, in "Report of the Mississippi River Commission for 1894," p. 2816.

case it would seem that the Arkansas City discharges were nearer to the truth than those observed at Wilson's Point. That a meter should register too high a velocity, from cross-currents, etc., is credible enough; but how can it register too low a velocity? Only from incorrect rating, one would suppose; yet the ratings of this particular instrument are remarkably accordant; indeed it was a newer and better meter than the one used at Arkansas City. If the recorded discharge at Wilson's Point is indeed too small, it is due to some cause yet unknown. Perhaps an eddy section was subtracted in the belief that the eddy was general when it was only superficial. Perhaps the reading of the meter at a submersion of 0.6 differed from the mean velocity in the vertical by a greater proportion than usual. Whatever the cause, it was general, for the error, if it be an error, is persistent.

Faults of Method.—Whatever method of determining velocities be used, its correctness depends upon an assumed relation between the velocities at different depths in vertical planes and the mean velocity in the same planes. By the method of double floats, and also by the ordinary or stationary method of meter observations, the velocity is taken at a depth which is a certain fraction of the total depth, usually 0.5 or 0.6. By rod floats, in great depths, the velocity of the mean of a certain number of filaments to a depth of 35 or 40 ft. is taken, and reduced by a formula depending on the curve of vertical velocities. In the flanking method the meter is kept at a constant depth below the surface, which depth bears, of course, a perpetually varying ratio to the total depth; so different factors are used at the different proportional depths to reduce to the mean of the vertical.

It was announced by Humphreys and Abbot, as one of their discoveries,* "that the ratio of the mid-depth velocity to the mean velocity in any vertical plane is practically independent of the depth and the width of the stream, of the mean velocity of the river, of the mean velocity of the vertical curve, and of the locus of its maximum velocity. In other words, it is a sensibly constant quantity for practical purposes." Their conclusion, supported by the figures which they bring to bear, is that at depths varying from 86 to 27 ft. the ratio has an extreme range only from 0.9868 to 0.9798.

In 1882, the parties who took the discharge observations made a large number of velocity measurements in vertical planes, which are

* "Report upon the Physics and Hydraulics of the Mississippi River," p. 311.

published in the "Report of the Mississippi River Commission for 1884," pages 2691–2723, tabulated for each tenth of depth from surface to bottom. The observations are more accordant than might have been expected from the difficulties of such work, with new men and insufficient appliances and at great depths. A few of the figures presented seemed to be anomalous, from misprints, errors or simply from unusual conditions.

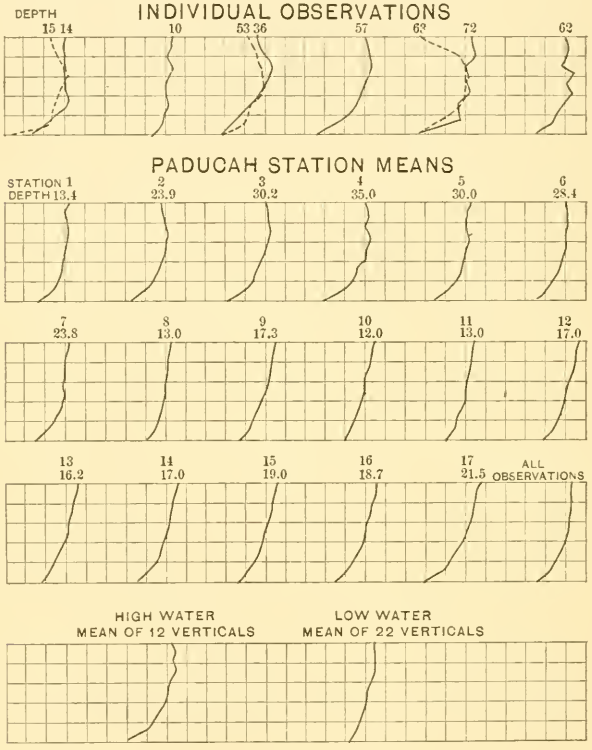


FIG. 6.

Plotting the Curves of Vertical Velocities.—In using these observations, the curves may be plotted which are developed by the velocities at different depths at the several stations, namely, Paducah, Columbus, Helena, Hays's Landing, and Red River Landing. In order to compare the curves with one another, the convenient practice of the engineers of the Mississippi River Commission may be adopted, and the depths expressed as fractions of the whole depth and the velocities as

fractions, proper or improper, of some standard velocity. The criterion generally adopted is the mid-depth velocity, partly because it is convenient, partly because it has been supposed to bear a close and nearly constant relation to the mean velocity.

When single observations are plotted, the curves are very irregular. They are not curves at all, in fact but broken lines. A few specimens are given in Fig. 6.

It would be a perplexing and, perhaps, an unprofitable task to attempt to unravel these intricacies. It would be better to take the means of different groups of these data, and try to eliminate in this

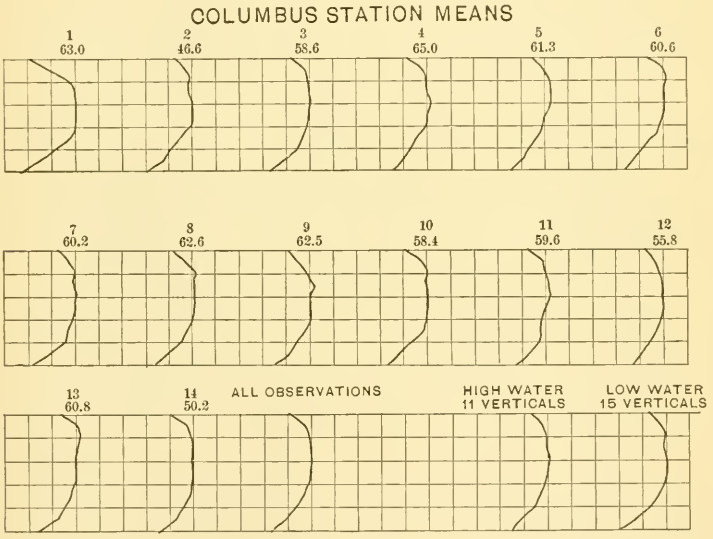


FIG. 7.

way accidental and local disturbances. The data of 1882 contain the "station means"; that is, the means of all the observations taken in each vertical plane throughout the season. At each of the five observation points a range was taken, as nearly as possible normal to the course of the river, and on that range a number of nearly equidistant stations were established, from 13 to 17 in number, from one bank of the river to the other. No. 1 was next the left bank. Observations were taken at these stations, in a more or less intermittent manner, from December, 1881, to November, 1882.

Plottings are shown in Figs. 6, 7, 8 and 9 of the velocity curves obtained by taking the means of the observed velocities at each station

during the year, at each of the observation points enumerated above, and also of the means of all the observations at each place. These results show a striking similarity in one respect, namely, in the resemblance between the curves of the different stations at the same observation point among themselves. At Columbus, for instance, the typical

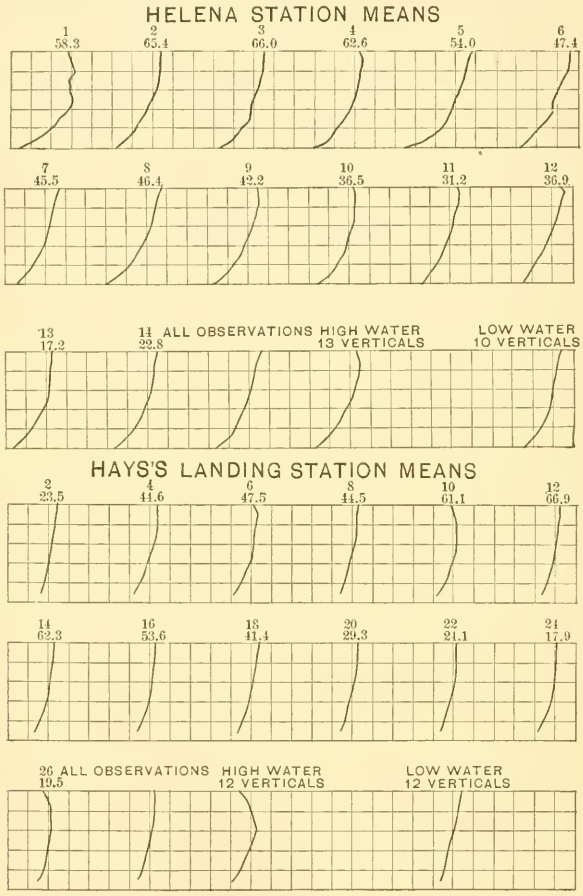


FIG. 8.

form of curve displayed by the combination of all the observations is distinctly traceable at each of the stations. At Paducah there is a little more diversity, owing to the peculiarity of the situation just below the mouth of a great tributary. Generally, the family likeness between the

station curves is very strong, and this notwithstanding differences in the depth of the water. At Helena, for example, the curve at Station 2, where the average depth throughout the season was 65 ft., is of the same type as at Station 13, with an average depth of 17 ft. So Hays's Landing, Station 12, with a depth of 67 ft., presents a curve which does not differ essentially from that shown by Station 24, with 18 ft. of

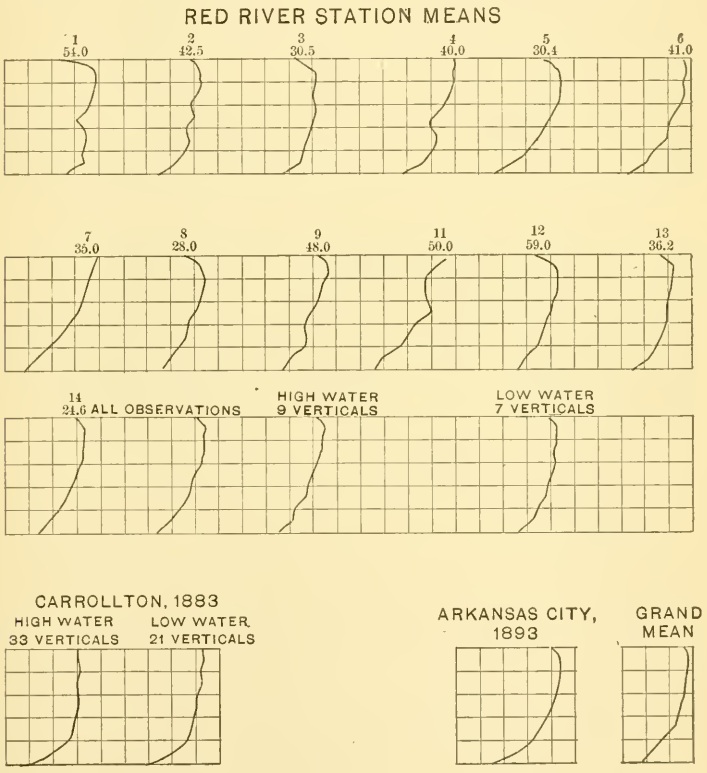


Fig. 9.

depth. One important difference there is, namely, that the minimum or bottom velocity often bears a closer ratio to the maximum in small depths than in great, a point to which attention may be directed hereafter. At Red River Landing the observations were so few (usually one or two at each station) that the means are very irregular, and are hardly more than individual observations. Fig. 9 also shows plottings

of Mr. Price's curve at Carrollton in 1883, and of Mr. Baily's at Arkansas City in 1893.

Ratio Between Mean and Mid-Depth Velocity.—The results of the observations of 1882, as far as they go, do not confirm the conclusions of Humphreys and Abbot with regard to the constancy of the ratio between mean and mid-depth velocity, nor as to the numerical value of that ratio. There are no mean velocities in the individual vertical planes reported. The ratio between the mean of the mean velocities in all the verticals at each observation point and the mean of the mid-depth velocities in the same verticals varied from 0.937 to 0.981. The mean of all observations is 0.958. At Paducah, Mr. Twining gives the mean coefficient for the reduction of mid-depth to mean velocity, in vertical planes, as 0.965, varying, however, for the several stations, from 0.944 to 1.000, and for the two sides of the river from 0.955 to 0.975. At Carrollton, in 1883, Mr. Price found the average percentage of mean velocity to mid-depth velocity to be 0.9625, being a little less for a rising river and a little more for a falling river. The range of values, for means, was from 0.9650 to 0.9571. Mr. Powless's observations at Fulton in 1879-80 were taken with imperfect apparatus (double floats), and seem hardly comparable to those of later date. They give a ratio of nearly unity, the mean velocity being sometimes the greater. The factor used at Arkansas City for reducing mid-depth to mean velocity is 0.9470. At Wilson's Point it is 0.9550. At Red River Landing the ratio 0.9572 is used.

The method suggested by Humphreys and Abbot for abridging the operation of taking discharges was to take velocities at mid-depth only, and to reduce the quantity thus obtained to mean, in each vertical, by the use of the coefficients found by them. As the relation, as announced by them, was so close, in the early observations under the Mississippi River Commission, namely, prior to 1883, the mid-depth velocity was assumed equal to the mean, and the discharges of 1879-80 and some of those of 1882 are computed on that assumption. In 1882, however, the velocities seem to have been obtained, in great measure, by the integration method. The reduction and discussion of the observations in vertical planes made in 1882, however, showed that this assumption was far from correct. At Paducah the mean velocity was found at 0.63 of the total depth, at Columbus at 0.71, at Helena at 0.61, at Hays's Landing at 0.60, at Red River Landing at 0.61. The

mean of all was 0.63. In Mr. Price's Carrollton observations of 1883, the mid-depth velocity was diminished by 3.5 to 4%, according to circumstances. In 1884-85 and subsequently, up to a recent date, the velocities were taken at 0.6 depth, and the results accepted as the mean velocities for those verticals without correction. At the present day a correction is sometimes applied, though not uniformly.

The method of ascertaining the mean velocities in verticals in 1882, whereby to compare them with velocities at different depths, was usually by integration, as practiced by Mr. Révy in 1871,* and, perhaps, by others; that is, the meter was lowered from the surface to the bottom and then raised from the bottom to the surface, the total number of revolutions being counted and the time elapsed being noted. The total number of revolutions divided by the time gives the mean velocity in revolutions in that vertical section.

Velocities Taken at Different Depths.—There are some objections to taking velocities at so great a distance from the surface as 0.6, particularly at high water, when depths are very great and currents very strong and turbulent, the meter under very imperfect control, and its behavior entirely exempt from scrutiny. Since a coefficient of reduction has to be applied anyhow, the innovation has lately been introduced of taking observations at various depths as might be most convenient, or at a fixed depth of 20 or 25 ft., as in flanking, and reducing by a factor. This method was used in 1893 at Arkansas City and Wilson's Point, and the results seem to have been more accordant and more satisfactory than usual. The factors of reduction used are given for the information of the profession. At Arkansas City: At 0.1 depth, 0.8978; at 0.2, 0.8966; at 0.3, 0.9081; at 0.4, 0.9272; at 0.5, 0.9470; at 0.6, 0.9831; at 0.7, 1.0411; at 0.8, 1.0964; at 0.9, 1.2339. At Wilson's Point: At 0.1., 0.9136; at 0.2, 0.9154; at 0.3, 0.9269; at 0.4, 0.9349; at 0.5, 0.9550; at 0.6, 0.9897; at 0.7, 1.0295; at 0.8, 1.0930; at 0.9, 1.1959. It would undoubtedly conduce to accuracy to use separate factors for great and small depths, from the fact that the curves of vertical velocities often vary much with the stage of the water.

Imperfections of Method.—The imperfections of the ordinary method are principally these: It is assumed that the 0.6 velocity, corrected or uncorrected, represents the mean velocity in any vertical plane, and it is assumed that the velocity varies uniformly from station to station in

* See his "Hydraulics of Great Rivers," pp. 17, 18, etc.

a horizontal plane. In the grand mean it is probable that these assumptions are nearly correct. In any individual case they are likely to depart widely from the truth. At Columbus, in 1882, the mean velocity was found at a depth of 0.71. At Carrollton, in 1883, at high water, the place of mean velocity was at a depth of 0.77; at low water, 0.66. It cannot be safely assumed, then, that the place of mean velocity is to be found at the same depth at all stations or at the same station at different stages; nor does there seem to be any good reason why such a result should be expected. The curves of vertical velocity, as plotted from the individual observations of 1882, differ widely from one another. Some show the mean velocity at 0.3 depth, some at 0.8. The error introduced by assuming 0.6 is sometimes as much as 0.5 ft. per second. It is usually much less than this, not more than 0.1 ft. The error consequent on the assumption of uniform variation of velocity in a horizontal plane will, of course, depend on the number of observation points. Some experiments were made by Captain Townsend, in 1893, with a view of determining the probable error caused by taking stations too far apart. Discharges were accordingly calculated with the stations 150 ft. apart, and afterwards with the intermediate stations omitted, making the points where the velocity was measured 300 ft. apart. The results are given on page 2949 of the "Report of the Mississippi River Commission for 1894." The greatest difference found was about 3 per cent. Ordinarily it was less than 1 per cent. The excess was not uniformly on either side.

It is evident that in any individual case it is a violent assumption that a single observation, at no matter what depth, will represent accurately the mean velocity of a partial area of 2 000 to 20 000 sq. ft., and that 20 such observations will give the mean velocity of a turbulent and irregular river of a cross-section of 200 000 or 250 000 sq. ft., and a discharge of 1 500 000 cu. ft. The errors may compensate one another, or they may be cumulative. In short, it is clear that anything like minute accuracy in the measurement of the actual discharge is out of the question by the means heretofore employed. It is also clear that very gross discrepancies in individual discharges should not cause surprise.

It is very possible, however, that the results attained by present methods are more accordant than they would have been if the actual discharge had been correctly ascertained; for they partake of the

nature of mean results, and are thus approximated somewhat to one another. From what has been already learned, it will appear that the actual discharge itself varies considerably from day to day and from moment to moment. It is influenced by the supply from tributaries and the return flow from basins, which is of itself very irregular, depending on accidents of rainfall, etc., by winds, by disturbances of cross-section due to caving and the passage of sand waves, by changes in the conformation of the shores, by the shifting of currents due to the change from low-water to high-water channels and the reverse, and doubtless to many other causes not understood or suspected. Ebbs and flows are frequently observed in the reading of gauges. Even on still days the gauge will fluctuate a tenth or so while the observer is looking at it. The existence of what have been called pulsations of the current has been affirmed and denied. In the case of winds, it is known that by their influence the thread of the current is shifted both vertically and laterally. In fact, every record gives evidence of what are called great irregularities in the motion of the water. Several instances of this kind have already been cited. It is true that these correct themselves in time, the slope being the great regulator; but this cannot prevent oscillations; it can only keep them from becoming excessive. As an instance of extreme departure from mean conditions, on May 3d, 1892, at Arkansas City, the measured discharge was 1 532 875. Next morning it was found to be 1 733 676. Astonished at this enormous discrepancy, the engineers repeated the observation on the afternoon of the same day, and found 1 741 526. The next day, May 5th, it was found to be 1 589 172. At Wilson's Point, 93 miles below, there is no trace of such a perturbation on that or the several following days. It was therefore momentary or altogether illusory. In 1893, at Arkansas City, an excessive discharge, as found by the ordinary method, was corrected by the flanking method.

Results.

The thirteen published volumes of the "Reports of the Mississippi River Commission" contain a vast mass of heterogeneous data on the subject of discharges, carefully reduced and tabulated in a suitable form for use. They cover pretty nearly every stage and every condition, and constitute a great storehouse of information. On attempting to make use, however, of these laboriously acquired facts, the investigator is

met at the very outset with a grave doubt whether they are facts or not. Many of them, as will be found on the most superficial examination, are flatly contradictory of one another. Therefore they cannot all be true. This being the case, which are true and which are false?

It is found that a discharge of 1 400 000 passed one place and of 1 200 000 passed another at only a day's interval, for two weeks together; that a discharge which passed last year at a gauge reading of 41 passes this year at 35; even the different seasons of the same year will witness an almost equal disparity. A conclusion drawn from the careful study of the observations at one place is completely upset by the teachings of another. The coefficient of the formula varies from 145 to 95 at the same place and stage, and within two weeks. The discharge curve is like a skein of tangled thread.

The first feeling of the investigator who plunges into this mass of perplexity is one of profound skepticism. There is no truth in it; or if there be any, it is impossible to distinguish it. Gradually, however, he finds that certain features are constantly reproduced; that even in the most complicated forms there are shadowy outlines, like those of composite photographs. Even the most chaotic-looking curve has a general direction, a tendency toward a central line or axis. He finds that what he rashly pronounced to be lies are often profound truths, concealed or deeply overlaid, it may be, by films of error, or distorted by temporary perturbations. What he wants is there, if he can only get at it.

Of the long list of discharges given in Table No. II, two series are particularly valuable; those of 1882, from their great extent, both in range of river and in time, and from the fact that they contain the records of the greatest known flood; and those of 1884–85, because they also cover a wide range from high to low water, and because they have been thoroughly discussed, and there is a pretty accurate knowledge of the methods practiced and the probable sources of error in them. Next to these in importance are the high-water observations of late years, in which it is possible to study the effect of the closer confinement of the flood waters and the greater heights thereby attained. The observations of 1888–89, while extending over several months, embrace only a limited range, being confined to perplexing and indecisive medium stages.

Objects Sought.

The immediate object is to find out, first, what was the greatest actual discharge that ever passed down the Mississippi, not only that which was held in the channel, but also that which passed in the swamps and sloughs outside of the banks; and, second, what height would such a flood have reached on the gauges of the several stations if it had all been restrained between banks or levees. Better than all would it be to find out the general law regulating the relations of the discharge to the gauge, so that it would be possible, not only to ascertain the height corresponding to the maximum discharge, but to any discharge, at any time, past, present or future. This last quest is the philosopher's stone of the river engineer.

The Flood of 1882.—In the year 1882 occurred what is considered the greatest flood that has ever been known, and if the discharge of that year had been measured with accuracy, half of the question would have been answered. But only a part of the discharge was measured, and that pretty crudely. In that year, the levee system was in a very imperfect, almost rudimentary, state. The Saint Francis Basin was not leveed at all. The White River Basin had only a few fragments of old and private systems. The upper portion of the Yazoo Basin had the remains of an old system in a ruinous condition. The lower part of the same territory had a continuous line, but weak and insufficient. The upper Tensas District (from Cypress Creek to a few miles below the boundary between Arkansas and Louisiana) had a broken and fragmentary line, as good as none. The Louisiana levees were small, weak and discontinuous. Consequently, long before the river had reached its height, it began to overflow its banks and to escape through gaps in the old embankments and frequently over their tops. As it rose still higher, it began to break the levees, and, finally, on the well remembered night of February 28th, a violent storm arose, which demolished the embankments of the lower Yazoo District in 30 or 40 places, and those of Louisiana in perhaps as many more. This was eight or ten days before the flood would have attained its culmination.

Of the vast volumes of water which escaped from the main river, large quantities found their way back to the channel. Of that which was abstracted immediately below Cairo, a considerable part returned in the bends immediately below, and was measured at Columbus.

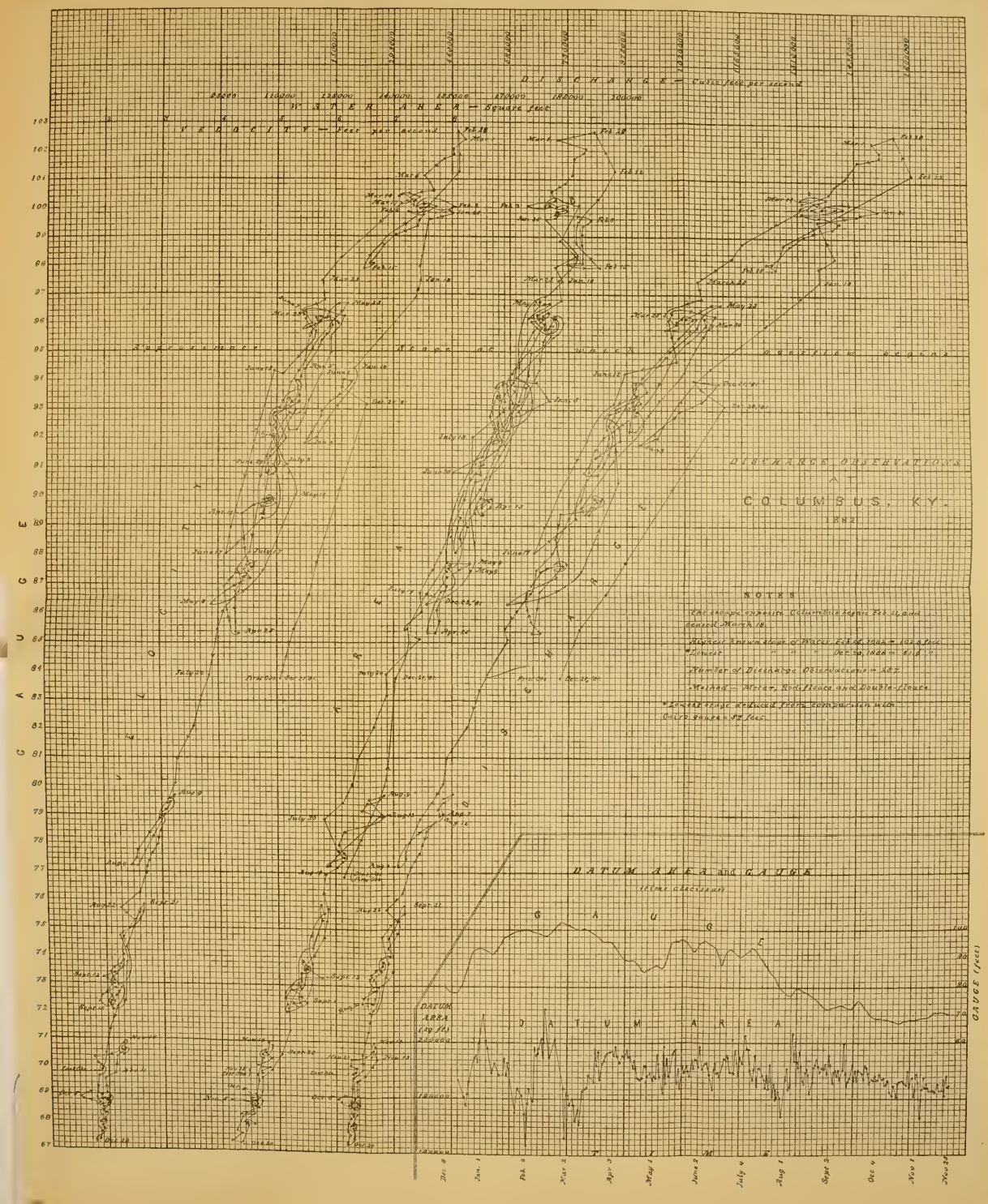
Nearly all came back into the river at New Madrid, but no discharges were taken there. Of that which poured into the Saint Francis Basin, all returned at or above Helena, except what was lost by evaporation or leakage or was retained in the lakes and swamps. That which escaped into the Yazoo Basin was returned, in the same manner, above Vicksburg. Of that which filled the White River Basin part was restored through the White and Arkansas Rivers, or at the foot of the basin above Arkansas City: part went to swell the enormous volume discharged into the Tensas Basin, to be partially returned through Red River, but mostly never to see the Mississippi again, being conveyed by the Atchafalaya and the low lands of the basin of the latter directly to the Gulf of Mexico.

Attempts were made to measure the escaping water as well as that which was retained in the channel, and in one way or another all of it was accounted for. It is very difficult to estimate the degree of credence to be attached to the crevasse and over-bank discharges. High-water discharges are attended with many sources of error, some of which have been indicated. Not only are the estimates of the volume of water passing liable to be deceptive, but the disturbances produced by crevasses, escapes and return flows have a very great effect upon the stage. Admitting, for instance, that 1 500 000 ft. of discharge passed Helena, yet it is probable that in a closely confined river it would have passed at a stage of 50 ft. instead of at 47. The behavior of a river while within its banks is comparatively regular. The discharges at such stages have been observed many times and at many places, and may be checked by one another or means taken. Observations made at such stages therefore afford the best means of studying the laws regulating the discharge, and the results arrived at by such reasoning may be applied, with proper modifications, to the case of a confined river at higher stages.

Discharge Curves.

To arrive at a preliminary conception of the law governing the progression of any series of data, a customary and excellent method is to plot those data to some argument that seems to have a connection with them and is itself known or easily ascertainable. This is an especially well recognized mode of procedure as regards discharges and velocities. Text-books say that discharge curves may be thus

PLATE VI.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXIV, No. 765.
STARLING ON DISCHARGE OF THE MISSISSIPPI.



constructed in which each successive gauge height shall correspond to so many cubic feet of discharge.

These curves, says Debauve,* are parabolas, convex to the axis of ordinates (the gauge heights). The curve of mean velocities is concave toward the same axis. These, he continues, are precious indications as to the general form of curves of discharge and velocity, and they enable these curves to be traced with great accuracy as soon as two or three points have been determined carefully, and the correctness of a series of experiments to be checked, and anomalies and errors to be detected.

The Discharge Curves of the Mississippi Very Complicated.—Debauve's remarks are in the main judicious; and doubtless in smaller rivers, and especially those of fixed regimen, it is comparatively easy to trace such a curve. In the case of the Mississippi, such a process is attended with many difficulties. Not only are the data very discrepant, but, even with perfectly correct numerical estimates, the relations between gauge and discharge vary extremely from time to time, owing to the instability of the bed. There are always, in all rivers, variations in these relations due to the rising and falling stages; but with the Mississippi this is but the beginning of complications. As an illustration, let there be plotted the curve of 1882 at one of the stations, say Columbus. The result is shown on Plate VI.† The figure has a general trend, but as to attempting to fit any particular curve to it, or deducing from it any law, it is baffling in the extreme.

These bizarre-looking figures may be greatly simplified by selecting for plotting only such observations as were made at stationary stages, and thus eliminating the effects of the rising and falling movements. The adoption of this course will give an assemblage of points, through which a curve may be drawn that will fit them fairly well. Indeed, half a dozen such curves may be drawn, none of which will fit perfectly, but all tolerably, and all about equally well; but when an attempt is made to prolong them, to compare the relations which would exist at high stages, it is found that the upper branches will depart widely one from another.

The curious and complicated sinuosities and entanglements of the actual velocity curves are not inexplicable. Part of them are due to

* "Navigation Fluviale et Maritime," p. 53.

† From the "Report of the Mississippi River Commission for 1883."

errors of observation. All of them are more or less affected by such errors; but the greater part, by their persistency and evident connection, show that they are genuine. There is no use in attempting to average them away. On the contrary, they are worthy of careful study, and an investigation of them cannot fail to be profitable. For this purpose, and in order to be able to discriminate between errors and unexpected or unexplained truths, it will be well to go back a little to first principles, and this the more that there is not an axiom of hydraulics which has not been called in question at the present day.

First Principles.

Mean Velocity.—The mean velocity of a stream is an artificial quantity, not actually existing in nature. It is simply the quotient obtained by dividing the discharge by the area of the cross-section.

The fact that the mean velocity is an abstraction is often lost sight of. Such expressions as "the water prism passing such a point" are often heard, the water prism being supposed to be a mass of water having a base equal to the area of the cross-section at that point and a length equal to the mean velocity for the time considered. In reality no such water prism passes. There does, indeed, a quantity of water pass equal in volume to such a prism, but it passes in fillets, each of which has a different velocity from that which adjoins it. The upper layers of each vertical section pass at a speed which is frequently double that of the bottom layers, and the surface velocity in the main channel is sometimes four times that next the shore. Grave errors have often been committed, are committed every day, by inattention to these facts, palpable as they are.

Slope of the Water Surface.—If there is any one principle in hydraulics that one would think firmly established, it is that in open channels the motion of each particle of water depends on the slope of the water surface. Yet it has been disputed, not so many years ago, by an engineer of reputation,* who thought that the observed phenomena of the flow of rivers could not be explained by it. He accordingly returned to the doctrine of the older Italian hydraulicians, that velocity in a running stream depended on depth below the surface. Indeed, these

*Robert Gordon, M. Am. Soc. C. E. See his "Theory of the Flow of Water in Open Channels." Rangoon, 1875.

phenomena are so puzzling that one may be excused sometimes for a little heterodoxy.

The postulate on which all formulas for velocity rest is this: In uniform motion the accelerating forces are equal to the resistances. The application of these formulas to living streams depends on the greater or less approximation of the actual circumstances to those demanded by the hypothesis of uniform motion.

Permanent and Uniform Flow.—The essence of permanent motion of water is equality of discharge at the various cross-sections. Uniform flow is defined to be that particular case of permanent flow in which the sections are equal and the bed of the channel parallel to the water surface. If it be conceded that in a flowing stream, after a greater or less time, a condition is attained similar to that demanded by the postulate, then an equation may be found which shall express the equilibrium between the accelerating and the retarding forces. The accelerating force is gravity. The retarding force is supposed to be friction, either of the flowing liquid on the solid walls between which it runs, or of the particles of liquid among themselves, the latter constituting what is commonly called viscosity or viscidity.

Considering only the former element, it is thought probable *a priori*, and it has been approximately found by experiment that the resisting power of friction of solids on liquids varies directly as the square of the velocity.

Laws of Fluid Friction.—The mass of evidence on which rest the so-called laws of fluid friction is very small—incredibly small considering the importance of the subject and the momentous conclusions which have been drawn from the said laws; and their correctness cannot be considered firmly established. The laws are as follows:*

The frictional resistance is independent of the pressure between the fluid and the solid against which it flows.

The frictional resistance of large surfaces is proportional to the area of the surface.

At low velocities of not more than 1 in. per second the frictional resistance increases directly as the relative velocity of the fluid and the surface against which it moves. At velocities of 0.5 ft. per second and greater the frictional resistance is more nearly proportional to the square of the relative velocity.

* "Encyclopedia Britannica," Vol. XII, p. 482, article "Hydromechanics."

Formula.—From these laws and the laws which govern bodies falling under the influence of gravity it is easy to deduce a formula depending on slope, cross-section and wetted perimeter, or, in short, slope and mean depth. The result is the well-known expression,

$$v = c\sqrt{r\,s},$$

which embraces in itself pretty nearly all the hydraulic formulas which have been attempted to be devised for rivers and for ordinary velocities. In the Chézy formula c is supposed to be a constant. In most of the others it is a function of r or of s, or of some quantity not included in the formula, as the roughness of the channel, etc. It must be remembered that it refers strictly to uniform motion, and that if it be attempted to apply it to any other case it must be modified to meet its new requirements.

General Formulas.

There are several reasons which militate against the employment of this formula, in any of its modifications, for general purposes; that is, as applied to all sorts of rivers, all stages, all slopes, all forms of section. The objections apply, also, to the use of the formula for different stations on the same river; and some of them are adverse to the possibility of devising, for the Mississippi and rivers of similar type, a rational expression for the relations between the several variables, even for different stages at the same place. This does not exclude the hope of finding empirical expressions which may be useful for local and temporary purposes.

Objections to General Formulas.—First.—The hypothesis of uniform motion is not even approximately realized.

Second.—It is asserted and taken for granted that the stream is always in a state of equilibrium between the resisting and accelerating forces.

Third.—The formula is founded on the supposition that the only resistance encountered is that of fluid friction, which is subject to the laws cited above.

Fourth.—It is assumed that the symbol v represents the mean velocity.

Fifth.—It is assumed that the bed of the stream, even though irregular, is unalterable. It will be seen that this is very far from true,

and that the changes in the area of the cross-section have a serious disturbing effect on the velocity.

Sixth.—For practical purposes, the only expression for the slope that is available is the slope taken over long distances, such as is estimated, for instance, from one gauge station to another. In the application of a formula, it must be assumed that this slope is the same as the local slope, s, of the formula, or bears a definite relation to it, which is not correct, unless by accident. Even the local slope is not constant for all parts of the cross-section, but varies greatly from bank to bank.

It will appear more clearly how very far the actual condition of the river departs from uniform motion. It is said, however, that the hypothesis may be applied on the supposition that the portion of the stream considered is so short, unobstructed and regular that it shall conform nearly to the required condition. This would confine the formula, strictly speaking, to one stretch at a time : or, at most, its use would be limited to localities which were exactly similar.

Equilibrium.—It is usually asserted and taken for granted that rivers are in a state of equilibrium, and this is the foundation of the theory of uniform motion. If a stream of water flow down an inclined plane, the velocity is accelerated by the action of gravity at a uniform rate. The resistance of friction, on the other hand, increases as the square of the velocity. Consequently, as the velocity increases, the resistance increases more rapidly, and a time comes when the two are equal ; and after that the motion is uniform, the work of gravity being expended in overcoming the friction of the bed.

Dubuat's Experiments.—It is well known that most of the experiments on which is founded the present knowledge of the motion of water were made on a very small scale. The greater part of Dubuat's famous series of observations were made with troughs less than 20 ins. wide, with a depth of water not exceeding 1 ft. It is wonderful what solid, valuable and even brilliant results were achieved with such imperfect means. It is almost equally wonderful how many inferences hastily drawn from such imperfect data (not, however, by Dubuat himself) have been accepted as facts. It is no doubt true that, given a channel of infinite length and unerodable and unchangeable bed, no matter what the cross-section or what the inclination, at some time a state of equilibrium would be reached. The smaller the stream, the

sooner would this condition occur. With very small channels it may take place within a few feet. In Bossut's experiments, made with a wooden trough 656 ft. long, even with so great a slope as 1 in 10, equilibrium was attained within the first 108 ft.* In very great rivers it may not occur at all. In fact, it is probable that it does not occur. If it did, then no matter what the relations between the resistance and the velocity, the equilibrium, being established upon a certain slope, would continue to subsist upon the same slope.

In fact, the equation $R = W \sin. i$, which expresses the equality between the resistance and the effective component of the weight, or the accelerating force, is impossible except in strictly uniform motion; that is, when the cross-sections are not only equivalent in area, but are similar in form, with the water surface parallel to the river-bed. The equation merely expresses the proposition that in a stream of uniform bed, section and slope, in which a condition of equilibrium has been attained, the velocity will be expressed by the relation $v = c\sqrt{rs}$. If any one of the conditions fail, the equation ought not to, and will not, hold good. If it be true of one shape of cross-section, it will not be true of another shape; if it be true of high-water conditions, it will not necessarily be true of low-water conditions; that is, in these cases c will not be a constant, or will not be the same constant at Helena as at Carrollton, at high water as at low. If it be true of a stream in which equilibrium has been reached, it will not be true when that condition has never existed or has been disturbed.

Modified Equilibrium.—Though the ideal equilibrium of the formula does not exist (for that would imply uniformity of slope), yet that there is a quasi-equilibrium is evident from the fact that the high-water velocity is nearly the same, in the mean, from the head to the foot of the alluvial valley; in other words, the accelerating and retarding forces balance one another in the long run. The accelerating force is always gravity; but the effective component of that force varies widely with the perpetual changes of local slope. The retarding force, too, is always of the same nature. It is the resistance of the bed; but this, in a natural stream, is very different from the supposed friction of a uniform and regular section in a straight reach with a fixed inclination. It is rather composed of a series of shocks or paroxysms, each followed by a recovery and a subsequent accelera-

* D'Aubuisson, "Hydraulics," p. 116. (American edition, by Bennett.)

tion. The plunge down a steep slope, *A B* (Fig. 10), is followed by a shock against the bottom at the change to a flatter inclination. The swift impulse in the nearly straight part of the bend *A B* is followed by the shock against the bank in the bight of the bend *B C*. Each of these shocks brings the velocity to a check, which is immediately succeeded by an acceleration where there is a renewal of the conditions favorable to the same. There is a series of oscillations. Each acceleration speedily brings about its own correction, which is rather an over-correction; and each retardment produces the conditions of a subsequent acceleration.

In a stream thus organized, a system of general equilibrium may be said to be established, but it must always be borne in mind that the expression is true only in a limited and qualified sense, and that the equilibrium is liable to disturbance whenever there is a change of circumstances.

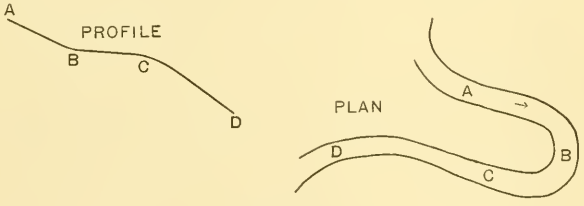


Fig. 10.

The formation of the bed of a sedimentary stream depends on the combined influences of all the different stages. Of these, the high-water stages are by far the most powerful, from both volume and velocity. The living force at extreme flood is more than a hundred times what it is at extreme low water. At bank-full stage it is twenty times as much as at ordinary low water, and five times as much as at medium stages. If it is assumed that in every 10 years there occur two very high waters, five reaching bank-full stage, ten in which mid-stage prevails and ten of ordinary low water, and if extreme high water be supposed to occupy one month, bank-full stage two months, mid-stage four months, and low water three months, the respective formative forces of the several stages for the 10 years will be about as 12, 50, 20 and 4; the great forces which prevail in extreme flood being partly compensated by the rarity and the short duration of the latter; and the frequency and length of ordinary floods and medium stages giving them an influence to which their dynamic importance would not

entitle them. On the whole, it is clear that the stages greater than the medium control the general features of the bed, and that the lower stages subject it to certain modifications which, in turn, are removed at the next flood.

There is no necessary antagonism, in general, between the high-water and the low-water forces. The general slope and general cross-section of the stream are not affected by the transition. It is not clear that any general fill or general scour is to be attributed to either stage. The modifications which occur from changes of stage appear to be local. They consist mostly in the building of the bars and the deepening of the pools in time of flood and the reversal of these processes in low water.

Changes of Water Slope with Stage.—While, by these operations, the general dimensions and slope of the bed are not affected, unless by an unusual prolongation of the high or low stage, or an unusual succession of high-water or of low-water years, yet the local modifications of them are considerable; and more than this, the water slope, which in flood exhibited an approximation to uniformity, and a certain independence of minor inequalities of the bed, in low water conforms more strictly to those inequalities, and great variations of local slope are introduced. The general equilibrium is not disturbed, or rather, there is substituted for it a different sort of equilibrium. There is no longer the violent shock against the banks or bottom. It is not necessary, and the conditions are not in force to produce it. The equilibrium which now exists approximates more nearly to the ideal equilibrium of uniform motion, with the necessary modifications due to violent changes of cross-section and slope.

If Equilibrium Exists at High Stages, It Will Exist at Low.—Let there be given a certain channel of uniform dimensions and of fixed slope, and let it be conceded that after a certain point has been reached a stream as full as the channel will bear will be in a state of equilibrium, the resistances balancing the acceleration. Let now the stream be reduced in size; it will still be in equilibrium. No matter how low the inclination of the channel nor how small the stream, if the greater volume has been passed, the less will be. But the resistance of friction will be disproportionately great, the velocity disproportionately low, and the area of the cross-section consequently increased. It will take a shorter time for the small and low stream to reach equilibrium

than for the large and rapid one. According to Dubuat's experiments, when equilibrium was established, the velocity then attained was equal to that due to the entire fall, and the head due to the mean velocity was equal to the difference between the height of the reservoir whence the canal was supplied and that of the uniform section.* The bottom of the reservoir and that of the canal were at the same level.

If, therefore, a channel be of such dimensions and slope as to carry a large stream in equilibrium, it will carry a small one. The resistance due to the area of the rubbing surface will be greater in proportion to the volume carried in the latter case than in the former, but the resistance due to velocity will be less. If the wetted perimeter be reduced from 4 000 to 2 000 and the volume from 1 500 000 to 100 000, then the proportion of rubbing surface to volume will be increased 7.5 times. If the velocity be reduced from 6 to 2.2 ft. per second, then the resistance due to velocity will be reduced 7.5 times. In short, the proportions will be so adjusted that a new equilibrium will be established.

Experiments on a Small Scale Not Necessarily Applicable to Great Rivers. —The conclusions drawn from experiments on a small scale cannot be applied, except by analogy, to great rivers, or perhaps even to small ones; and in applying the analogy, due heed must be paid to all circumstances. It cannot be inferred, for instance, in the case of the Mississippi, supposing the origin of velocities to be Cairo, that an equilibrium would be attained as soon as a mean velocity of 6 ft. was reached, the fall due to which is 0.56 ft., corresponding to about a mile and a half of distance. If such were the case, the slope, as has been intimated before, would be constant below this point. But would not a curved slope carry the discharge as well? Undoubtedly; but in the case of equilibrium, the velocity, under the circumstances of a decreasing slope, would be reduced and the cross-section increased as the slope became flatter and flatter, which is known to be the reverse of the truth, and which would, indeed, be incompatible with the very existence of a silt-bearing stream, as it would not permit the transportation of its load of sediment. Such a condition would lead to the gradual and pretty rapid filling up of the lower reaches of the river.

The means by which the velocity is maintained nearly constant under the continual changes of slope are several. In that part of

* D'Aubuisson, "Hydraulics," p. 136.

the river where the slope is steepest, the cross-section is wide and shallow. Bars and islands are plentiful. Large volumes of water are lost over "points" in flood time, or escape into the great basins and are returned at the outfalls of the same. There are violent alternations of width, deep bends, frequently with banks of inerodable or difficultly erodable material. As the slope becomes flatter, the cross-section becomes more favorable to the passage of the discharge, deeper, narrower, until, as the Gulf is approached, it has not much more than half the width that it possessed above Helena, and has almost double the depth. It is also somewhat straighter and has much easier curves and a more symmetrical and more uniform cross-section.

The resistances above enumerated are very difficult to be reduced to a formula. While it may be said that the sum of them is equal to the accelerating force, yet how is the sum of them to be obtained? Resistances of this kind cannot be classified under the head of friction. The assumption cannot be made of them that they are independent of pressure or weight. They partake, in fact of the nature of work.

Fluid Friction and Viscosity.—It has been seen that the resistances to the flow of water are of two classes; one depending on the solid surface with which the water is in contact, and the other on the internal relations of the particles of the fluid among themselves, and these have been called by the names of friction and viscosity. If these terms are to be understood as embracing between them the total resistance, they must be accepted in very different significations from those which they usually bear. It is evident, from what has just been said, that pure friction is responsible only for a part and possibly a small part of the resistance of the walls of the channel; and it appears certain that pure viscosity cuts a very insignificant figure in the internal resistances.

If the motion of water took place with perfect regularity, if the sides of the channel were perfectly smooth and the layers or laminæ slid on one another without disturbance, the effect of viscosity would be comparatively slight. This fact was adverted to by Dupuit, in his discussion of the relations between bottom and mean velocity.*

Says Professor Unwin:†

"Boussinesq has shown that the central filament in a semi-circular canal of 1 m. radius, and inclined at a slope of only 0.0001, would have

* "Mouvement des Eaux," p. 55.

† "Encyclopedia Britannica," Vol. xii, pp. 459–460.

a velocity of 187 m. per second, the layer next the boundary remaining at rest. But before such a difference of velocity can arise, the motion of the fluid becomes much more complicated. Volumes of fluid are detached continually from the boundaries, and, revolving, form eddies traversing the fluid in all directions, and sliding with finite relative velocities against those surrounding them. These slidings develop resistances, incomparably greater than the viscous resistance due to movements varying continuously from point to point. The movements which produce the phenomena commonly ascribed to fluid friction must be regarded as rapidly or even suddenly varying from one point to another. The internal resistances to the motion of the fluid do not depend on the general velocities of translation at different points of the fluid (or what M. Boussinesq terms the mean local velocities), but rather on the intensity at each point of the eddying agitation. The problems of hydraulics are therefore much more complicated than problems in which a regular motion of the fluid is assumed, hindered by the viscosity of the fluid."

Indeed, many of these phenomena are not dependent on viscosity at all, for movements of an analogous nature are found even in air, such as eddies, whirls, and even the drawing-on of one stratum or layer by the friction or attraction of the one adjacent.

Dupuit's Objection.—It has been denied that the formula gives the quantity sought, for the reason that the v of the formula does not represent the mean velocity, but the actual velocity of that particular part of the stream that rubs against the bank. It is to the square of this velocity that the resistance is proportional, and it is this quantity that enters into the equation.*

The force of this objection seems to be somewhat diminished by the fact that in the experiments on which is based the knowledge of the laws of fluid friction, the velocity measured was not that actually existing at the surfaces of contact between the solid and the liquid bodies. Most of these experiments were made by moving solid surfaces through liquids, and the velocity which was proportional to the resistance was the velocity of the surface relatively to the banks, not relatively to the velocity of the envelope of water immediately adjacent to the solid, which participates in the movement of the solid. It may be fairly asserted that a part of the resistance experienced by the moving surface consists in imparting this motion to the surrounding water against the cohesion of the adjacent particles; and that conse-

* Dupuit, "Mouvement des Eaux," pp. 4-5.

quently the friction between the solid surface and the film in immediate proximity to it is not the friction sought, nor is the relative velocity of the two the velocity sought. In like manner, when the liquid is in motion and the solid stationary, the same reasoning may apply; that is, the bank of the river has already done a part of its work in retarding the velocity of the fillets immediately adjacent to it, and of those more remote fillets to which the influence of the adjacent fillets extends; and that hence the velocity of those retarded fillets should not be the criterion of the resistance experienced. If it were, then the velocity of the layer immediately next to the bank should be the one considered. Now this is nothing, or next to nothing.

Velocity of the Formula not the Mean Velocity.—There is undoubtedly, however, a flaw in the reasoning by which the velocity of the formula is made to appear to be the mean velocity of the whole section. The velocity and the resistance to which it is proportional vary at every point of the perimeter. At A, Fig. 11, the velocity is 1 ft.; at B it is 5 ft. If the perimeter be divided into n nearly equal spaces, then (supposing the proportion constant) the resistance in each part of the perimeter will be equal to $Al\frac{p}{n}v^2$, and the total resistance will be equal to $Alp\frac{(v_0^2+v_1^2+v_2^2+\ldots v_n^2)}{n}$, where A represents the constant of proportion, l the length of the stretch of river considered, and p the wetted perimeter. Now the mean velocity is equal to $\frac{Bv_0+Cv_1+Dv_2+\ldots Nv_n}{B+C+D+\ldots N}$, where B, C, D, etc., represent the partial areas. In the ordinary way of misunderstanding the formula, the total resistance is made equal to $Apl\left(\frac{Bv_0+Cv_1+\ldots Nv_n}{B+C+\ldots N}\right)^2$ instead of to the expression first given above. In the one, v^2 is taken to mean the average of the squares of all the partial or local velocities at every point of the perimeter; in the other it represents the square of the sum of the products of each individual velocity by its partial area, divided by the square of the total area. This error is not always of great consequence at high stages, but with particular forms of sections may

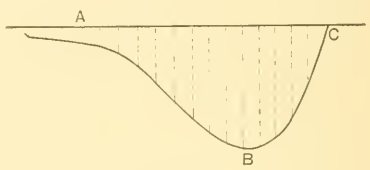


FIG. 11.

become very important. In one instance (Helena, in 1884–85), it amounted to 0.12; in another (Warfield's Point, 1893), to 0.14; but in still another (Bachelor's Bend, 1893), taken on the same day as the preceding, and only a few miles distant, to 1.31; all this at mid-stage, with a discharge of about 700 000.

It is well known that there is great difficulty in applying the Chézy formula or any formula to streams of different depths or even to the same stream at the same station at different stages of water. Observation shows that in such cases c is not a constant at all, but varies with the stage and with other circumstances. Will Dupuit's supposition afford a relief from this difficulty?

Ratio Between Mean and Bottom Velocities.—The relation between the mean and the bottom velocities at different stages is not well understood. The ratio between the maximum, mean and minimum velocities has often been given in the text-books as about 5, 4 and 3, irrespective of depth. It was inferred by Dubuat, from his experiments, that the ratio of velocity at the surface to that at the bottom is greater according as the velocity is less, and that this ratio is entirely independent of the depth. He also observed that the mean velocity was a mean proportional between the surface and bottom velocities. This question has been made the subject of an elaborate investigation by Dupuit* who deduces the following formula:

$$\text{Bottom velocity} = \frac{\text{mean velocity}}{1 + 0.73r}.$$

If the ratio between the mean and bottom velocities be nearly constant at all stages, it makes little difference which signification be attached to the symbol r. The only difference between the use of the mean and the bottom velocity is that the coefficients will be less in the latter case, but they will differ among themselves by the same ratio.

Observations of 1882.—There are not many data from which to form a decisive opinion with reference to the relation between the mean and the bottom velocity at different depths. The observations of 1882 furnish a mass of information, some of which can be made to bear upon this point, or rather as to the relation between maximum and bottom velocity. By combining the different verticals, there will be found for this relation:

* "Mouvement des Eaux," p. 55.

At Paducah.

At depths above 60 ft.....	5	observations,	0.47
" between 50 and 60 ft....	5	"	0.49
" " 40 and 50 "....	6	"	0.56
" " 30 and 40 "....	21	"	0.56
" " 20 and 30 "....	61	"	0.63
" " 15 and 20 "....	27	"	0.67
" " 10 and 15 "....	62	"	0.74
" below 10 ft.....	24	"	0.76

At Columbus.

At depths above 60 ft.....	55	observations,	0.65
" between 50 and 60 ft....	33	"	0.64
" below 50 ft.....	12	"	0.60

At Helena.

At depths above 60 ft.....	19	observations,	0.58
" between 50 and 60 ft....	23	"	0.53
" " 40 and 50 "....	15	"	0.54
" " 30 and 40 ". ..	13	"	0.58
" " 20 and 30 "....	15	"	0.68
" below 20 ft.....	5	"	0.71

At Red River Landing.

At depths between 50 and 60 ft....	6	observations,	0.64
" " 40 and 50 "....	4	"	0.66
" " 30 and 40 "....	10	"	0.49
" " 20 and 30 "....	8	"	0.66
" below 20 ft.....	4	"	0.57

At Carrollton, in 1883, the mean of 33 high-water verticals gave a ratio of bottom to maximum velocity of 0.55. A mean of 21 low-water verticals gave a ratio of 0.49.

These results are not at all accordant, and there is not much profit in combining them further; especially as there is no direct method of comparing either the bottom or the maximum velocity with the mean. At Carrollton, Mr. Price found the ratio of mean to bottom velocity at high stages to be 0.58, and at low stages 0.53. However, Carrollton is

subject to very peculiar low-water conditions. For the whole series of observations, the ratio between maximum and bottom velocities is about 0.66, which corresponds very closely with that found by M. Bazin. Dupuit's formula gives, for depths of 60 ft., a ratio of mean to bottom velocity of 0.69; for a depth of 20 ft., 0.87.

This result M. Dupuit admits with great candor is not in accordance with experience, and he attributes the discrepancy to the internal movements of the particles above referred to.

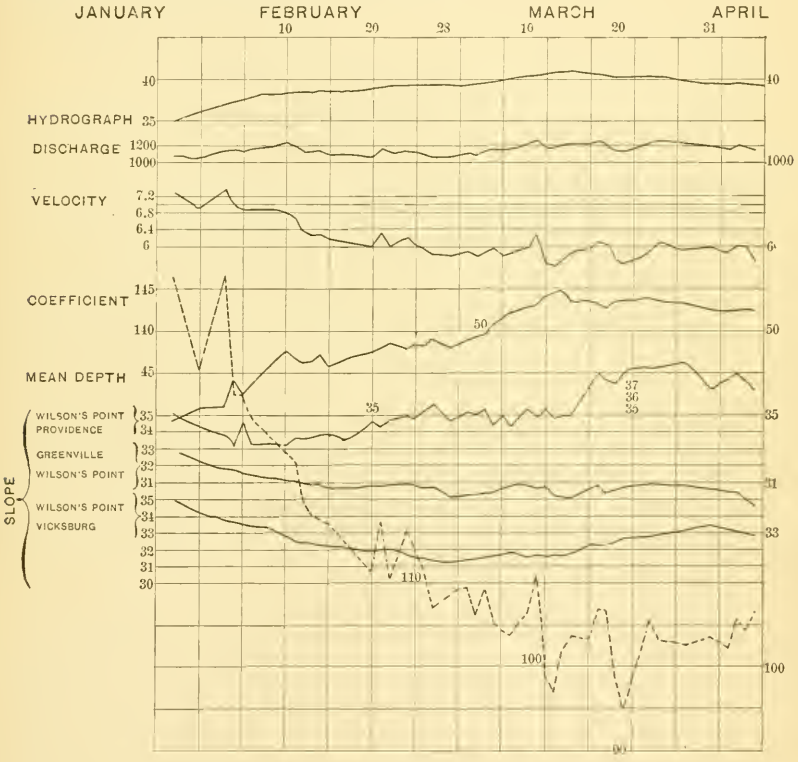


FIG. 12.

It cannot be affirmed, then, that the substitution of the bottom for the mean velocity would bring the formula into any closer accord with facts.

Variations of Cross-Section.—The variations in cross-section which progress during each season are accompanied by disturbances of the velocity which are not only unrecognized by the formula, but are apparently in direct opposition to it.

In any formula of velocity which has ever been proposed, if the slope remain constant (other circumstances being unchanged) and the mean depth be increased, the velocity will increase. Yet the observations of every year, at every point, show instances where increase of depth is accompanied by loss of velocity, not only where other circumstances seemed to be unchanged, but where they apparently conspired to aid the increased depth in bringing about an augmentation of the velocity. A remarkable example of this kind is given in Fig. 12, which is a plotting of the high-water discharge of 1890 at Wilson's Point. The phenomena are perfectly explicable, and, indeed, are perfectly natural, but they are not at all in accordance with the formula.

The theory of permanent motion demands as its first requisite uniformity of discharge. Indeed, it is impossible that streams could continue to flow, as experience proves they do flow, on any other supposition. The discharge that passes *A* must be, on the whole, the same that passes *B*, excluding, of course, disturbances from rises or falls. If it were otherwise, if at *A* there passed even 10 000 cu. ft. per second more than at *B* for 24 hours continuously, there would be an accumulation between *A* and *B* of 864 000 000 cu. ft.; and if these stations be 40 miles apart there would be a rise throughout the reach of 1 ft. Experience shows that the movement of the discharge takes place with remarkable uniformity. At stations which are contiguous, or which are even hundreds of miles apart, provided no tributary or outlet intervene, the gauges move from day to day with almost parallelism, subject only to the gradual oscillations of slope which accompany rises and falls.

Incompatibility of Conditions.—But if the discharge be uniform and the cross-section vary, the velocity must vary also, but in an inverse sense. The greater the area, the less the velocity. In the instances cited the increase of area was all in the direction of depth, the width remaining constant. Therefore, the condition of permanency of discharge seems to be diametrically opposed to the fundamental postulate demanded by the formula, the increase of velocity with mean depth.

Variations of Local Slope.—While the general slope does not vary materially during these operations, yet it is probable that the local slope experiences considerable changes. It will be seen in many instances that local slope is constantly working and is of great power. It is certain that the local slope corresponding to a high velocity can-

not be maintained when the velocity is considerably lessened. Information on this point is very scanty, and, indeed, the difficulty of defining what constitutes local slope and of ascertaining it after it has been defined, and of separating it from eddy action, etc., is very great; nor is it apparent that for the present purpose the benefit derived from such observations would be commensurate with the trouble involved in making them. In truth, at any given point, the influences of velocity of approach and of local slope are inextricably intermingled; and it is probable that the relative importance of the two elements is frequently interchanged at the same station.

LOUISIANA BEND, APRIL 9, 1890.

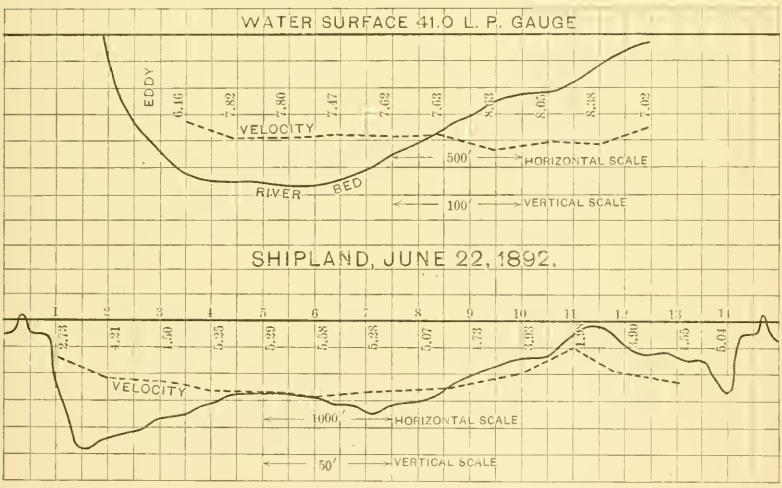


FIG. 13.

Mean Depth at any Particular Station not always Controlling.—The dictum that mean depth has an immediate controlling influence on velocity is frequently contradicted by the experience derived from measurements of relative velocity in various parts of the cross-section. It is true that the contradiction is more apparent than real, and that the variations of depth in such cases are often or always compensated by opposite changes of local slope. If the assumption that mean depth is controlling be absolutely true, it should hold good, not only for different sections of a river, but for different parts of the same section. But it is found that the swiftest current is not always in the deepest water, even in the same section. This fact is clearly shown in the

plots of velocity observations. These are not usually published, but an instance may be cited, from sources accessible to examination, in the observations taken in Louisiana Bend in 1889, 1890 and 1891,* and at Shipland, in Mississippi, on June 21st and 22d, 1892.† Selections from these are reproduced in Fig. 13. In both these instances it will be observed that the strongest current is not uniformly or usually in the deepest water, even in flood time, but is sometimes in comparatively shallow depths.

In the former case, an explanation of this phenomenon is readily afforded by the knowledge that the local slope is widely different in different parts of the cross-section. The observations were taken at the extremity of a "point," in the bight of a deep bend, and most of them were taken at high stages. The slope next the bank in the deep water was very small at such times, in accordance with a general law. Across the point of the bar it was very steep. The rapid descent at the latter place more than compensated for the shallowness of the water. The reach at Shipland is broad and of slight curvature. It cannot be doubted, however, that, owing to local causes, the threads of the current that possessed the highest velocity had also a very steep local slope. The same phenomena are presented in the Helena observation of March 20th, 1885 (see page 410).

Complexity of Conditions.—It will be perceived that the complexity of local conditions may be such that it is almost impossible to unravel their intricacy or to deduce from them any general or average condition. What, for instance, is the local slope at Shipland or in Louisiana Bend? On the one bank it may be 0.5 or 0.6, or even 1, ft. to the mile; on the other it may be 0.05 ft.

The slope of the formula is the inclination of the stream at the point considered. This can hardly be regarded as a determinable quantity. Many attempts have been made to ascertain it by direct measurement, but without success. The fall per mile is so small that a trifling error in the levels will make a great error in the slope. It is very difficult to ascertain what is the exact water level, affected as it is by ebbs and flows, pulsations, etc., and it is well-nigh impossible to eliminate or to estimate the effects of eddies, cross-currents and threads of current. The local slope is not a simple

* "Report of Mississippi River Commission for 1893," Appendix 5, Plate 114.

† The same, Appendix 5, Plate 11.

matter at all. It has been shown already that there is reason to suppose that each part of the cross-section has a slope of its own, and further evidence will be adduced in support of the same opinion.

Velocity of Approach.—The doctrine of an established equilibrium, so generally accepted, has led to an almost total neglect of the consideration of the possible influence of the velocity of approach. After a state of equilibrium has supervened, this element is not to be considered at all. But if there be no such equilibrium, due account should be taken of it.

In the state of quasi-equilibrium, which may probably be considered the normal or usual condition of the river, velocity of approach often plays an important part in the adjustment of local conditions. The state of quasi-equilibrium consists, as has been stated, in a series of oscillations, of accelerations and checks. The general slope in flood time is too great for equilibrium; it is so great that, if equilibrium were established, the velocity necessary to bring it about would be fatal to the stability of the banks and bed. Therefore, whenever the river has a straight course for a few miles, its velocity is sensibly accelerated, until a corrective is applied by the deepening of the channel and the formation of a shoal or island below, by bends or by some other well known means.

It often happens that long pools occur, both at high and at low water, in which there is a strong current with less than the average slope. At high water, the mere appearance of a long pool with little slope may be deceptive; for at such a time the current is often not next the bank in the bend, but across points or bars, so that the bend approximates to the condition of a pool; but at low water the current hugs the bank closely. Now it is possible for a considerable velocity to be established at a low slope, but it is not possible that it should be acquired with no slope at all, or with a reverse slope. It is, however, possible that it may be maintained, once acquired, at such a slope under favorable circumstances for a limited distance.

Reverse Slopes.—In the series of observations made at Carrollton in 1879-80, under the direction of Captain Leach, and thoroughly discussed by him,* reverse slopes are mentioned as occurring persistently, and even at high water. Captain Leach does not consider it probable that this phenomenon could have been due to eddies, the latter, as he

* "Report of Mississippi River Commission for 1882," pp. 98-115.

remarks, being usually high-water phenomena, or being, at least, most vigorous at that stage and decreasing in activity with the decline of the river, whereas the reverse slopes were most decided at low stages. From the observations he draws the following important conclusions :

"But if it is admitted that the water was moving down stream, while at the same time its surface inclination was up stream, it must also be conceded that the momentum of the moving water enters as a factor in determining the conditions of its flow, and hence no exact relation necessarily or even probably exists, in natural channels, between the velocity of the water at a particular time and place, and the local slope of its surface at the same time and place. This principle may be applied to the sectional areas as well. By the term local slope is meant the fall of the water surface which would result from a measurement made over a very short, or relatively infinitesimal, length. Such measurement is, in the nature of things, impossible, and the quantities here discussed are not strictly local slopes; but the argument turns on the existence of reverse slopes, which being demonstrated for any distance, it follows, from the continuity of the surface, that reverse slopes must exist over infinitesimal distances at numerous points between those determined.

"If local slopes be rejected, it follows that an average fall over an assumed distance, greater or less, must be taken, as representing the slope which determines the discharge."

The evidence of the existence of actual reverse slopes, however, cannot be considered indubitable. The surface of the river, as is well known, is a warped surface, and the current crosses from one side to the other, producing great disparities of slope on the two sides.

General and Local Slope.—In determining the coefficients, in the discharges of 1890, and, indeed, in all the reductions, the slope has been taken between the discharge station and the nearest gauge above and below. The slopes so ascertained have differed materially from one another as they were taken above or below the station. Thus the slope Bolivar-Arkansas City, in 1892, was 0.000044 when the slope Arkansas City-Greenville was 0.000063. The coefficient deduced from the one was 109 when that deduced from the other was 91. Nevertheless, there was a pretty regular relation between the two systems of coefficients. At Wilson's Point, in 1890, it was possible to get an approximation to a local slope by comparing the gauge at Lake Providence, about 10.5 miles distant. The results differ somewhat from both that deduced from Greenville, above, or Vicksburg, below, which are more accordant among themselves. They show the maintenance, with considerable

persistency of a local slope decidedly greater than the general slope either above or below. After the occurrence of a great crevasse below Lake Providence, the local slope increased considerably.

It is not possible that a velocity should be maintained indefinitely by momentum upon a slope which could not originate such a velocity. The resistance, R, whether the stream be in equilibrium or not, is proportional to plv^2; in other words it is the same for a given rubbing surface and a given velocity, irrespective of slope. The resistance will therefore be as great for a small slope as for a great one, provided the velocity be the same. The acceleration, other things being equal, is dependent on the slope. Therefore, under the supposed circumstances, there would be an acceleration which is less than the resistance, which would throw the stream out of equilibrium and cause a rise, contrary to the hypothesis. If the slope be too small to originate such a velocity as may be prevailing at any place, the resistance will gradually overcome it and lessen it.

The fact that a greater coefficient is required in the lower reaches of the river, where the slope is flatter, may be explained by admitting that the slope in this part of the valley is itself sufficient to confer and maintain the requisite velocity, under the favorable circumstances existing there, and that the coefficient of 150 or more required there is the normal coefficient for that form of section and trace of bed. In the upper reaches, where the conditions are very unfavorable, the velocity is reduced by shocks, bends and other circumstances which do not enter the formula, and the coefficient is less.

The distribution of the local slope in a transverse sense is shown by the published notes of the Helena discharge of March 20th, 1885.* Table No. III shows the partial areas, the observed velocity for each area, the partial discharges and partial mean depths. It shows also the partial coefficients, obtained by the application of the formula $v = c\sqrt{rs}$ to each partial area. For this purpose the slope will be assumed the same for all, being that actually prevailing on that day, as estimated from the nearest gauge station, Mhoon's Landing, 29.2 miles above. If, instead, the slope is taken from Helena to St. Louis Landing, 43 miles below, the only essential difference will be that the coefficients will be greater, but they will bear the same relation among themselves. The assumption that the slope is the same in all parts of

* "Report of the Mississippi River Commission for 1887," p. 2825.

the cross-section is more likely to be true at a medium stage, like that presented in the example, than at a very high stage; yet it is not at all correct even here, as will presently appear. In the table the areas are 200 ft. wide, except at the ends, where they are 261 and 126 ft. respectively. They are measured between half-stations, the velocities being taken at the whole stations, given in the table. Thus, the first area extends from Station ½ to Station 1½, plus 61 ft., etc. Areas are placed opposite the velocity stations, which are in their middle.

TABLE No. III.—Helena Discharge, March 20th, 1885.

Station.	Partial area. Square feet.	Velocity. Feet per second.	Partial discharge. Cubic feet per second.	Partial mean depth. Feet.	Partial coefficient.
1	9 084	3.51	31 885	34.8	64
2	12 415	4.30	53 385	62.1	58
3	11 945	4 97	59 367	59.7	69
4	10 637	4.86	51 696	53 2	71
5	10 090	5.00	50 450	50.4	75
6	9 570	5.10	48 807	47.9	79
7	9 420	5.12	48 230	47.1	80
8	8 598	5.05	43 420	43.0	82
9	7 720	6.01	46 397	38.6	104
10	6 982	5.32	37 144	34.9	96
11	6 140	5.26	32 296	30.7	102
12	5 570	5.13	28 574	27 8	104
13	5 065	4.22	21 374	25.3	90
14	4 940	4.30	21 242	24.7	91
15	4 890	3.72	18 191	24.5	93
16	4 978	4.14	20 609	24.9	80
17	4 940	3.88	19 167	24.7	84
18	4 815	3.82	18 393	24.1	83
19	4 902	3.80	18 628	24.5	82
20	5 045	3.50	17 658	25.2	75
21	5 320	3.19	16 971	26.6	66
22	5 343	3.05	16 296	26.7	63
23	4 875	2.95	14 116	23.9	64
24	1 461	1.54	2 250	11.6	48

Total area, 164 655 sq. ft.; velocity, 4.473 ft. per second; discharge, 736 546 cu. ft. per second; mean depth, 34.4 ft.; coefficient, 81.64.

From this table it is apparent that the coefficient increases very regularly, excepting one or two discordant observations, to the middle of the river, and then decreases, not quite so regularly, to the other bank. It seems to bear no relation whatever to the depth, and but little to the velocity. The coefficient at 3.5 ft. per second is the same as at 5 ft. It is probable, then, that the increase of the coefficient toward the middle of the stream depends on local slope. There is a "crossing" just below Helena, that is, a place where the current leaves one bank and crosses over to the other. It is well known that at such places there is often a great difference of elevation of the water surface

on the two sides, amounting, as has already been explained, to a considerable fraction of a foot, or even to a foot. It is probable that such a condition existed at this point, the thread of the current having a slope greatly in excess of the general slope of the river in this vicinity. Such instances are common, and mention has been made of other parallel cases.

Resistance at the Surface.—The formula of Humphreys and Abbot differs from all the rest in substituting for the hydraulic mean depth or hydraulic radius a "mean radius prime," obtained by dividing the area of the cross section by the wetted perimeter plus the width. This was because they found at the surface a resistance analogous to that experienced at the surface of contact between the water and the sides of the channel.

If the velocity in rivers were affected only by the friction of the bed, it is evident that in a straight and symmetrical channel the swiftest current would always be found in the parts most remote from the banks on one hand and from the bottom on the other. If the course of the stream were crooked and its cross-section unsymmetrical, the place of maximum velocity would no longer be in the middle, but it would still be at the surface. But it has long been known that the greatest current is not always or generally found in this situation. It is evident, then, that there is a force other than the friction of the bed which is at work, and this force exercises a retarding effect upon the water at the surface.

It is sometimes supposed and asserted that Humphreys and Abbot ascribed this retardation to friction against the air. This is altogether erroneous. They did indeed consider the resistance of the air as partly the cause of the retardation, but not wholly or mainly. The retardation was attributed by them principally "to the loss of living force arising from upward currents or transmitted motion occasioned by irregularities of the bottom. This loss is greater at the surface than near it."* So again:

"Enough has now been learned to justify the remark that the resistance at the surface in calm weather can be only partly due to the friction against the air; otherwise a down-stream wind, moving with equal velocity with the water, must reduce it to zero and raise the axis to the surface—a result contrary to the observations. It occasions no surprise, to one familiar to the boils and whirls of the Mississippi, that

* "Report on the Physics and Hydraulics of the Mississippi River," p. 302.

they should cause a great loss of living force at the surface, and consequently a great retardation of the surface current."*

The views thus advocated are substantially the same as those put forth afterwards by Professor James Thomson:†

"He points out that portions of water, with a diminished velocity from retardation by the sides or bottom, are thrown off in eddying masses and mingle with the rest of the stream. These eddying masses modify the velocity in all parts of the stream, but have their greatest influence at the free surface. Reaching the free surface they spread out and remain there, mingling with the water at that level and diminishing the velocity which would otherwise be found there."

It is a particular case of the reasoning previously quoted from Boussinesq.

All that was claimed by the famous American investigators was‡ that there was a retarding force of some sort at the surface, and "that this resistance, whatever its cause may be, is of the same order or nature as that at the bottom and sides of the channel, since the law of transmission through the fluid is the same in each case." That is, they held that the film of water next the air moved more slowly than the layers adjacent to it below, and hence exerted a retarding influence on them, and through them on the whole mass. The error they made was in attributing to this resistance the same importance as to the resistance of the bed, whereas it is not one-tenth of the latter. This error is not of great consequence. If there were absolutely no resistance at the surface, and the expression p, in the equations, for the wetted perimeter, were arbitrarily taken at double its real value, the area, a, remaining the same, it is obvious that the only effect on the resulting formula would be to reduce the value of the mean depth by one-half, and hence to increase the coefficient c.

Local Formulas.—In the face of these objections it would seem to be extremely difficult to arrive at a general formula adapted to channels of all sizes and conditions. It is very difficult to devise even a formula which shall be suited to large rivers, or to an individual river like the Mississippi, at all slopes, places and stages. But there is a chance of success in the search for local formulas, adapted to particular places and perhaps particular stages.

---

* "Report on the Physics and Hydraulics of the Mississippi River," p. 271, note.

† "Encyclopædia Britannica," Vol. XII, p. 497.

‡ "Report on the Physics and Hydraulics of the Mississippi River," p. 317.

General Considerations.

Some general considerations may be premised.

Nature of the Resisting Force.—The resisting force, if it be not in the shape of friction, is in the shape of work. Now, the resistance of a rough bank and bottom, so far as it is not purely frictional, is merely the interposition of obstacles to the free flow of the current. To overcome these obstacles requires work. It is not unanimously conceded that the resistance developed by a stationary obstacle to the motion of running water is the same as that presented by still water to the movement of a solid body through it. The preponderance of opinion, however, is very largely on that side. The identity or close analogy of the two cases is tacitly conceded when the results of Colonel Beaufoy's or of Mr. Froude's experiments are applied to the case of resistance of the banks. It is conceded whenever a current meter is rated by the usual method.

The resistance of a surface which is perpendicular to the direction of the current is approximately equal to the weight of a prism of water whose base is the surface and whose altitude is the height due to the velocity. The resistance of a surface which is inclined to the current is much less, according to the degree of inclination and also according to the form of the surface ; the coefficient of difference varying, in extreme cases, from 1 to 0.22. The water presses on both sides of the surface. In ordinary cases the difference of pressure on the upper and the lower side of a body, especially if it have both its faces inclined to the direction of the current, is so small that it may be neglected. In any event, the resistance is dependent on the square of the velocity and not on any other power.

The vertical and lateral disturbances in the nature of eddies and boils also are work, as well as all resistances except those of pure friction and viscosity, and possibly the latter also, if they be worthy to be taken into account. Hence it may be inferred that the resistances of all kinds are functions of the square of the velocity.

The velocity here mentioned is not the mean velocity, but is composed of velocities of all possible magnitudes between the maximum and that passing next the bottom. In the impossibility of finding any formula which shall express the relations between these elementary quantities of work, and consequently of the velocities on which they depend, it is necessary to rely on the mean velocity in hypothetical

cases, or on partial velocities so far as they may be ascertainable, regarded as means for the areas which they represent; trusting to correction, by empirical methods, of the errors introduced by this imperfection of method.

That the resistance of the bottom, other things being equal, varies with the extent of the rubbing surface, seems undeniable. That it varies in exact proportion to it, irrespective of shape or depth, cannot be asserted. Whatever errors are introduced by the assumption will show themselves in the coefficient.

Intricacies of Slope.—It seems to be impossible, with the available data, to unravel the intricacies of general and local slope, and the respective agencies of velocity of approach and of velocity due to fall. Usually nothing is known except the general slope. If the exact local slope were known for every filament of the surface, it is not certain that the investigation would be simpler. It is therefore necessary to accept the general slope for better, for worse, and make the most of it.

It will not be worth while to attempt to find a formula which shall be adapted to places and circumstances of both high and low slope, for the reason that there are important modifying conditions which are not cognizable by formulas at all, or, at least, by any which have ever been proposed. These conditions are the straightness or easy curvature of the river, symmetrical shape of cross-section, permanency of channel, smooth and mostly non-caving banks, etc. They prevail in the lower part of the river below Baton Rouge, and have a most important effect in modifying the velocity.

It is easily seen that in a straight and symmetrical channel, the banks and bottom of which are composed of unusually tough and hard material, the water will acquire a velocity much in excess of that which will prevail in a region full of sharp curves and with friable bed. Very little slope may be required to confer a considerable velocity where there are no obstructions. Now, the average curvature of the river is very nearly the same from Baton Rouge to Carrollton as from Natchez to Baton Rouge. The curves, however, are generally easier, the bottom is of harder material, the widths and depths more regular. There is not the same alternate expansion into shoals and contraction into pools as in the upper river. The river carries for the most part only its normal burden of silt, and its energy is not dissipated in transporting from place to place enormous loads of eroded

material beyond its ability to carry permanently. Now, the average curvature of two portions of a stream may be the same when the conformations, as to resistance, are widely different. Thus, in the two plans shown in Fig. 14 the ratio between the shortest distance, *A B* or *D E*, and the distance by river may be exactly the same; yet it is evident that the former trace will afford a far greater resistance to the passage of a current than the latter. Below New Orleans the river is almost straight, with the exception of the "English Turn."

The mere change of direction, if gradual, does not interfere with velocity. Thus the planets move in curved orbits without diminution of speed. It is the impact against resistances that causes the loss of energy, and hence of velocity; and this impact is more or less effective, according to the angle of presentation.

If the course of the river were straight, with a slope even as low as 0.1 ft. to the mile, the velocity would be more and more accelerated until it might become absolutely destructive to a bed composed of friable earth. In such a case it would scour out the bottom, produce an alternation of pools and shoals, and eventually bank erosion and the formation of bends. A bottom of more refractory material would allow of a higher velocity without suffering any alteration, and might even tolerate a current so swift as to bring about at that small inclination a condition of genuine equilibrium between the accelerating and retarding forces. In other words, it is perfectly possible for an inclination of 0.1 ft. to the mile at Carrollton to produce and maintain as great a velocity, under the circumstances existing at that point, as for an inclination of 0.4 or 0.5 ft. at Columbus or Helena under the peculiar conditions that characterize the river in those stretches. It is true that a part of the difference of circumstances appears in the ordinary formulas, under the form of mean depth; but only a part. The rest of the differences must appear, if any of those formulas be adopted, in the coefficient. As it is exceedingly difficult to devise a a general expression for the effect of the differences, it would appear

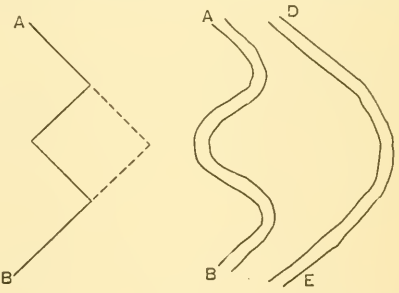


FIG. 14.

easiest and most satisfactory to find experimentally a coefficient for each place and perhaps a different coefficient for each stage.

Reduction by the Use of Coefficients.—This method is of constant adoption in the practical application of mechanics or even of mathematics. When the circumstances are so complicated as to defy the powers of analysis, a simpler case is assumed, the explanation of which is within reach, and the results of the rational formula deduced from the hypothetical case are compared with the phenomena shown by experiment. The two may bear a relation which shall be close enough to be expressed by a coefficient which is either constant, or variable according to a known law. If a series of parabolic areas are to be computed and the method of calculating parabolic segments is not known, in default of better methods they may be assumed to be rectangles. If the contents of one of the given areas is measured, by dividing it into partial areas or otherwise, the relation between it and the corresponding assumed area is found to be two-thirds, and this expression, applied as a coefficient to each of the assumed areas, will reduce it to the correct quantity. Most often the result is not so satisfactory, and frequently it is necessary to be content with a mere approximation and often with a coefficient that varies according to laws which are themselves obscure.

Causes of Discrepancies in Actual Velocities.

The great discrepancies found in the measurements of velocity are principally due to these four causes: (1) inaccuracy of data; (2) inapplicability of formulas; (3) changes of slope; (4) changes of cross-section; the last class being partly connected with the preceding or third class.

The first and second causes have been sufficiently discussed. Changes of slope are of several kinds. There are: (1) changes due to rise and fall; (2) changes due to stage of river; (3) changes due to alteration of section by scour and fill during the season; (4) changes due to permanent change of bed by secular causes.

The departures of the discharge curve from the regular figure which it is generally expected to present almost all exhibit considerable persistency, indicating the existence of law. During the progress of a rise it was observed that the curve uniformly shows a departure to the right, indicating a decided increase, and as decided a shifting to the

left or lower side during a fall. By comparing the curve of discharges with the curve of velocities (see Plate VI), it will be found that the principal features of the former are faithfully reproduced in the latter, while the variations of the area move sometimes in a direct and sometimes in an inverse sense.

Of the more regular movements of the velocity, the greater part are attributable to change of slope; the rest mostly to change of area.

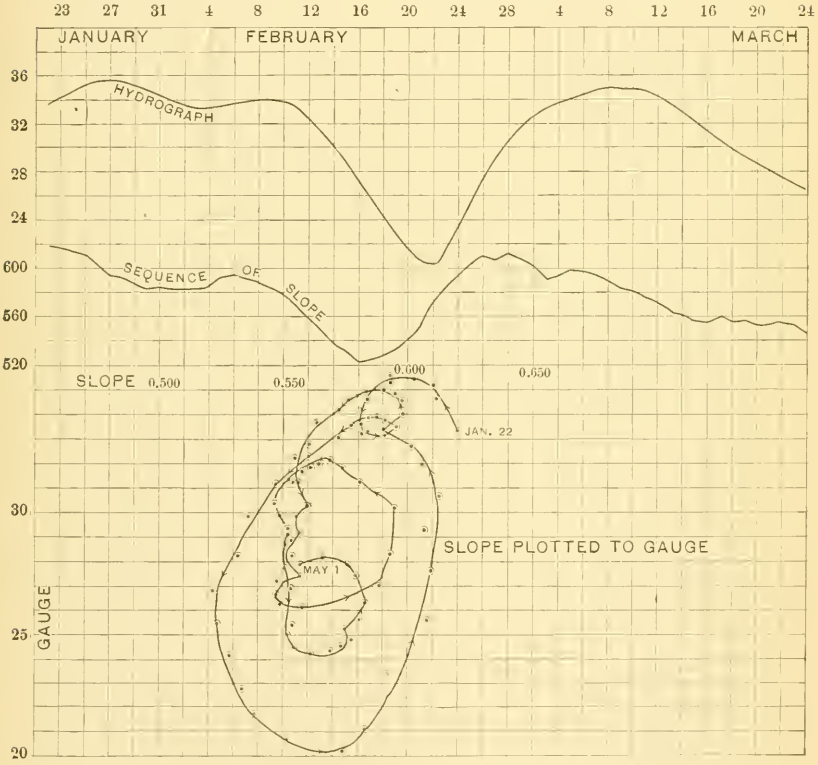


FIG. 15.

I. Changes of Slope.—Each accession of water from above, at any station, from the main river or from a tributary, causes an elevation of the surface at that station, the influence of which, at first, is confined to a limited distance. In other words, the rise at *A* is at first absolutely imperceptible at *B*. If the freshet be sudden and the stations 100 miles or so apart, there may be a rise of several feet at the upper station while the lower is unaffected. The slope, as estimated between these

stations, will then experience a sudden and great augmentation, which may progress at a still more rapid rate, may continue at the same rate, or may fall off. Sooner or later, all these alternations will take place in the course of every rise. The process which goes on during the fall is just the reverse of this.

The movement of the slope in rises and falls is shown in Fig. 15. taken from the gauge records of 1889. The points selected for comparison are White River Landing and Arkansas City, 43.9 miles apart. The first plot shows the movement of the slope from day to day, or what Colonel Suter calls a sequence of slope. The second shows the movement with reference to the gauge.

From both plottings it will be seen that in this distance there is no regular relation between the height of the river and the slope. The actual value of the slope evidently depends on the suddenness and magnitude of the freshet. This particular slope is much more liable to disturbance than most others, as White River Landing is just below the mouth of two large tributaries, with drainage basins which are the recipients of a large rainfall, often very fitful.

Effect of the Great Basins on Slope.—At high water, the fluctuations of slope are often very great from the disturbing influence of the great basins in subtracting water from the river at their upper ends and returning it at their lower ends. In former years this influence was much greater than it is at present. When the upper parts of the Yazoo and Tensas Basins and all of the Saint Francis and White River Basins were open, as they were in 1882, 1883 and 1884, as much as 300 000 or 400 000 cu. ft. per second were suddenly added to the discharge at Helena, Vicksburg and Red River Landing, raising the river with great rapidity at those points, greatly flattening the slope above them and steepening the slope below. In 1882, when the water which escaped over banks and through gaps and crevasses into the Yazoo Basin returned at Vicksburg, the river, which was then falling, began to rise rapidly (0.5 ft. per day); the slope from Hays's Landing to Vicksburg, which had been 0.27 ft. to the mile, dropped to 0.18 ft. and a less discharge passed Hays's Landing at 38.6 ft. on the gauge than had passed previously at 36.4 ft., or than passed afterwards at 34 ft.

Peculiarity of the White River Basin.—With the complete closure of the Yazoo and Tensas Basins, and the approaching closure of the other two, these perturbations will be reduced to such as are produced by the

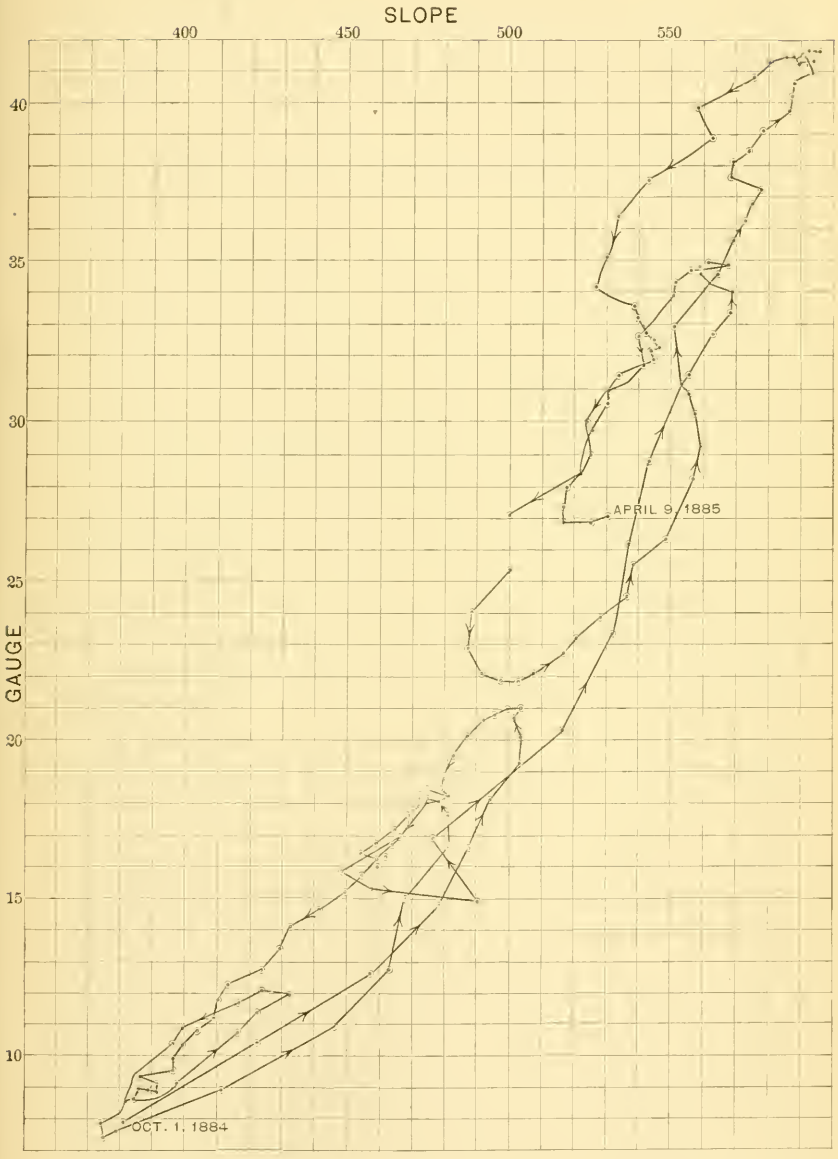

SLOPE
400
450
500
550
40
35
30
25
20
15
10
GAUGE
APRIL 9, 1885
OCT. 1, 1884

Fig. 16.

actual discharge of the tributaries, increased occasionally by the discharge from great crevasses, should such unfortunately occur. The condition of the White River Basin is peculiar in this respect, that at high stages a considerable part of the discharge of the two great tributaries is not poured directly into the Mississippi through the mouths of the White and Arkansas Rivers, but overflows the basin, flows through it parallel to the main river, and is discharged along the Cypress Creek levee, a few miles above Arkansas City, thus introducing a new element of disturbance into the slope relations.

Slope Arkansas City–Greenville.—Between Arkansas City and Greenville there is no disturbing influence, unless, perhaps, at high water a crevasse might occur. The gauges are 40 miles apart. A plotting is given in Fig. 16 of the curve of slopes between these stations, with the gauge height at Arkansas City as the argument, for the season of 1884-85. Here there is seen an increase of slope with height of river. The slope is much more regular than the slope White River-Arkansas City.

Movement of Gauges.—The increase of slope with gauge height, however, is not a feature which prevails everywhere. Indeed it cannot be, for such a relation would involve a progressive decrease of oscillation from the head of the alluvial valley downward, which is not true. The range at Cairo is about 1 ft. less than at Vicksburg, and a little more than at Arkansas City. The range at Greenville and Lake Providence is some 7 ft. less than at Cairo, and 8 ft. less than at Vicksburg. The gauges at Arkansas City and Vicksburg move very nearly parallel, so that if the slope Arkansas City-Lake Providence increases with a rising river, the slope Lake Providence-Vicksburg must diminish. Such in fact is the case. The following little table shows the extremes, taken from the high water of 1893, the highest recorded at Arkansas City, and the low water of 1894, the lowest ever known throughout this reach. As there were several crevasses in 1893, a correction has been applied to the readings of the Greenville, Lake Providence and Vicksburg gauges, to deduce from them the water level as it would have been if those disturbing influences had not occurred. For comparison, the high-water slope of 1888 is also given. In that year there were no crevasses.

Slope.	Low water, 1894.	High water, 1893.	High water, 1888.
Arkansas City-Greenville	0.178	0.340	0.323
Greenville-Lake Providence	0.332	0.322	0.327
Lake Providence-Vicksburg	0.437	0.302	0.307
Arkansas City-Vicksburg	0.331	0.319	0.319

In other parts of the river it is found that the same discrepancy prevails, the stations between Cairo and Helena showing a much less oscillation between high and low water than prevails at those points. At Memphis it is 16.2 ft. less than at Cairo, and 13.3 ft. less than at Helena.

Greater Oscillation at the Mouths of Tributaries.—It has long been known* that there was a much greater oscillation of the gauges at the mouths of tributaries than at intermediate stations. This is partly due to the greater discharge at the former places during great floods. For instance, at Cairo, the greatest discharge that was passed between banks in 1882, as reported, was about 1 500 000; at Fulton, about 1 250 000; at Memphis, about 1 250 000; at Helena, about 1 500 000; at Arkansas City, about 1 300 000; at Greenville, about 1 200 000; at Lake Providence, about 1 000 000; at Vicksburg again, about 1 500 000. These differences proceeded from loss of volume over banks unprotected by levees and through crevasses in the latter, and from the return of the water thus lost at the foot of the several basins. Had as great a discharge passed Memphis as passed Cairo, the water level at the former station might have been 4 or 5 ft. higher. This, however, only accounts for a part of the discrepancy. In 1885, when there was no overflow, the oscillation was, at Cairo, 30.7; at Fulton, 23.7; at Memphis, 25.1; at Helena, 32.7; at Arkansas City, 34.9; at Greenville, 29.7; at Lake Providence, 29.1; at Vicksburg, 37.5. The greater oscillation at Helena, Arkansas City and Vicksburg than at Cairo is partly explained by the greater volume of water which passed at those points by the accession of tributaries, growing continually larger as progress is made down stream, and sometimes reduced to almost nothing when the tributaries were very low; but the difference between Arkansas City and Greenville or Lake Providence, or the difference between Cairo and Fulton, cannot be thus explained. As has been shown in the "Report of the Mississippi River Commission for 1884,"† the difference seems here to be in the low-water mark; it is in the extreme fall and not in the extreme rise.

The proximate cause of the less oscillation of the river at intermediate stations is the profile of the river-bed, which is convex between the mouths of tributaries. Now the low-water slope conforms to the

* "Report of the Mississippi River Commission for 1884," p. 2550.

† p. 2550.

shape of the bed much more closely than does the high-water slope. The former, then, is flattened in the reaches between the mouths of tributaries and the intermediate stations, involving as an immediate consequence a decrease of velocities at these points and an increase of cross-section at low water. The data for the comparison are scanty, for not many low-water discharges have been taken. At Helena, in 1891, the extreme range of recorded velocities was from 6.47 to 2.24. At Fulton no extreme high-water discharge was taken. The highest recorded velocity was 7.01; the low-water velocity was as small as 1.51. At Memphis, the velocity varied from 8.65 to 1.40; at Wilson's Point, from 5.83 to 1.84; at Red River Landing, from 5.24 to 2.63.

The most striking example of a small oscillation is afforded by Carrollton, which is a suburb of New Orleans, situated at its upper limit. Here the extreme oscillation is only about 17.4 ft. The high-water velocity is usually something more than 6 ft. Very few low-water observations have been taken at Carrollton; but in 1891, the lowest velocity must have very little exceeded 1 ft. per second. The water area generally varies from about 190 000 at high water to 140 000 at low. In 1884–85, its extreme limits were 185 072 and 148 437; the velocity varying from 6.16 to 1.61. At Helena, during the same season, the areas ranged from 206 060 to 53 448, and the velocities from 4.96 to 2.82. Thus the discharge is passed at the lower station with a much less difference of cross-section between high and low water, and consequently with a much less oscillation in height. In the above instance the range of the gauge at Helena was from 40.62 to 7.17 = 33.45 ft.; at Carrollton, from 13.54 to 0.34 = 13.20 ft. The comparison is still more striking if the case of Red River Landing is taken, only 192 miles above Carrollton, and with exactly the same discharge. In 1884–85, the areas at Red River Landing ranged from 236 688 to 97 589, and the velocities from 5.08 to 2.25. The oscillation was 34.58.

It has been shown that the slope from Arkansas City to Greenville varies from 0.178 at low water to 0.340 at high. On the other hand, the slope from White River to Arkansas City at low water is 0.378; at high water 0.269 ft. to the mile. Thus, at Greenville the conditions are favorable to a comparatively low velocity at low water and a high velocity at high water, while at Arkansas City the tendency is in the opposite direction.

The Beds of Sedimentary Streams Self-Regulating.—The phenomenon of the convexity of the river-bed between tributaries is so general that it assumes the proportions of a law, and it demands an explanation. A plausible one is found, in the alluvial part of the river, in the effects of the variations of discharge alluded to above. It is regarded as a well established fact that the bed of a sedimentary river regulates itself according to the quantity of water it has to carry. Now, at the mouths of tributaries the discharge is much greater than at the intermediate points. The great volume here poured in has scoured out a suitable channel for itself, which has been gradually contracted down stream by the continual loss of water in floods over the banks. In the case of the Mississippi, the discharge of the tributaries has been greatly augmented, sometimes doubled, trebled or quadrupled, by the return flow from the great basins. Usually, the tributary which drains the basin is the principal channel through which the overflow water returns to the main stream, as in the cases of the Saint Francis and the Yazoo Rivers. The White and Arkansas Rivers, as has been noted, form an exception to this rule.

Causes of Contraction of Bed by Loss of Volume.—It is not necessary to discuss in detail in this place the process by which the bed is contracted by the loss of volume of water. In brief, the contraction is due to the slackening of velocity brought about by the disproportionate reduction of area in comparison with wetted perimeter, or in short by a diminution of mean depth. In the equation $r = \frac{a}{p}$, a is reduced more than p. A decrease of gauge height of 8 ft. means a reduction of wetted perimeter of only 16 ft., or 0.004, but of area of 32 000 sq. ft., or 0.133 of the whole. No doubt also some consequence is to be attached to the deflection of the thread of the current by reason of the lateral draught, though the degree of importance to be attached to this influence is still a matter of controversy.

The oscillations of slope due to stage must not be confounded with those caused by rise and fall. The latter are temporary disturbances of the ordinary relations, and may occur at any time. The former are of a permanent nature, and while not perfectly regular in their movements, yet, when dissociated from the perturbations of rises and falls, they exhibit a general law of direct or inverse progression relatively to the stage. This progression can be exam-

ined in the tables of discharges which have been selected for discussion, and in the plottings.

The table given on page 420 may be extended as follows, from the observations of 1891. This year was selected because it was not a year of very great flood nor therefore of great disturbance of high-water conditions, and also because it was a year of very low water.

Slope.	Low-Water Slope, 1891.	High-Water Slope, 1891.
Cairo-Columbus	0.196	0.310
Columbus-New Madrid	0.219	0.341
New Madrid Cottonwood Point	0.521	0.476
Cottonwood Point-Fulton	0.362	0.422
Fulton-Memphis	0.482	0.432
Memphis-Mhoon's Landing	0.439	0.486
Mhoon's Landing-Helena	0.547	0.328
Helena-Sunflower Landing	0.295	0.405
Sunflower-Mouth of White River	0.375	0.277
White River-Arkansas City	0.378	0.269

As has been stated, these movements are not at all regular. Sometimes, at low water, the slope White River-Arkansas City will be less than at high, as in 1884–85 and in 1889. At all times considerable perturbations are likely to occur in the transitions from stage to stage, from changes of cross-section, etc.

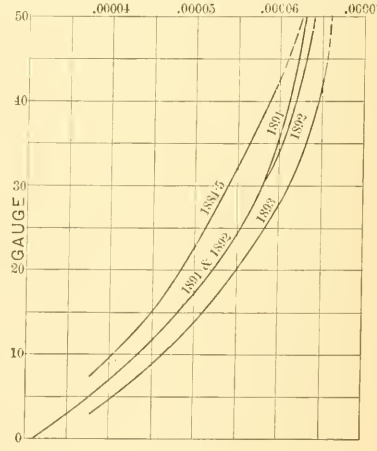


FIG. 17.

Secular Changes of Slope.—Beside these temporary or periodic changes of slope there are others that may be called secular, requiring a considerable time for their development, and of a permanent character. They are probably intimately connected with changes of the bed. Observations on the Mississippi have been conducted for too short a time to afford grounds for any accurate conclusions as to the effect of long-continued causes, enduring for a series of years, on the regimen of the river. It is probable, however, that the slope relations between different points have been materially modified since the beginning of the observations. A sketch is given in Fig. 17, showing the slope Arkansas City-Greenville, plotted to gauge, for the years

1884–85, 1891, 1892 and 1893, on the assumption that the distance between the two points remained constant. In order to eliminate the effect of rises and falls, the slopes have been taken only at times when the river was stationary, or at the culmination of a rise or the lowest point of a fall. It is to be observed that in many instances the change was so rapid that it is by no means certain that the exact slope at such times was ascertained. The observations are taken but once a day and generally to the nearest tenth of a foot, and on successive days the slopes are sometimes found to vary by 0.01 ft. to the mile, or 0.000002 of the distance.

Changes of Slope between Arkansas City and Vicksburg.—It would appear by an inspection of Fig. 17 that the slope Arkansas City-Greenville had increased materially in eight years, and by a steady progression applying as well to the low as the high stages. This phenomenon may indicate an enlargement of the cross-section at Greenville. It must be remembered, however, that the length of the river is perpetually changing. Accurate data as to the amount of the change are not forthcoming, but it is probable that the distance between Arkansas City and Greenville has increased since 1882 by about 2 miles. If this be true, the slope, in this instance, has really suffered little or no alteration.

II. Changes of Area.—The changes which the area undergoes are at least of three kinds. One is the daily alteration caused by the passage of sand waves, the caving of banks, and generally by incidents which may be considered of a temporary and fortuitous nature. Another is the grand annual periodic change due to the transition from high to low water or the reverse, whereby material is transferred from bar to pool, or from pool to bar, the general or mean bed of the river suffering no change. A third embraces changes which are permanent or of long period, such as those just alluded to.

Rapid Fluctuations of Area.—The daily fluctuations of the area are sometimes extremely violent. At Arkansas City, in 1885, a scour of 11 000 ft. in two days and of 14 000 ft. in three days is recorded, followed by a fill in the next two days of 8 000 ft. Changes of 5 000 or 6 000 ft. in a day are not uncommon. When it is realized that a mistake in sounding of a foot or two throughout the section would produce such an apparent result, there is a natural feeling of incredulity when these figures are read. The observers, however, who are usually educated

and skilful engineers, believe firmly in their reality. In 1892, at Wilson's Point, Mr. Miller reports, from Stations No. 4 to 6, where very large boils existed, a fill of 16 ft. in 24 hours.

It has already been remarked that these changes of area are generally accompanied by a change of velocity, and that this is a strong confirmation of their genuineness. There is no reason whatever why an error in sounding should have any effect on the indications of a current meter. If, therefore, a mistake were made in the measurement of a cross-section, the result would be perceptible in the shape of an increased or diminished discharge. If there be no such result, but the change of area is offset by a change of velocity in the opposite direction, the presumption is in favor of the reality of the phenomenon.

Passage of Sand Waves.—A number of very careful experiments have been made to determine the quantities of material moved along the bottom, or rolled in the form of sand waves, as they are generally called, and to measure the height, shape and direction of the waves, their rate of travel, etc., at Plum Point, Lake Providence and Carrollton during the season of 1879–1880. It would appear that these movements are sometimes of great magnitude, the crests of the waves being often as high as 22 ft., their slopes flat on the upper side, and steep on the lower. In 1880, Arthur Hider, M. Am. Soc. C. E., found that at high water, in the shallow sections, the average height was about 13 ft., the distance between crests about 450 ft., the rate of travel about 22 ft. a day. In the deep sections, the height of the waves at high water was about 7 ft.; length, 330 ft.; travel, 40 ft. a day. At lower stages the waves are not usually so high, the distances are shorter and the rate of travel slower; but the movements are still of considerable magnitude. Even in very shallow water there were found numberless small sand waves 1 ft. or 2 ft. high and less than 100 ft. apart. Now, the passage of a wave 1 ft. high would make a difference of area, in an ordinary discharge section, of 3 000 or 4 000 sq. ft.

Changes of Area Due to Alternations of Stage.—By far the most important changes of area, however, are those which take place during the transitions from high to low stages and the reverse; and it is in these changes that some of the most curious and characteristic phenomena of the river find their explanation.

Of the many varieties of cross-section found in the Mississippi as well as in other rivers, there are two extreme types, the narrow and the broad, pertaining respectively to the pools and the shoals. At

medium stages these do not differ very much in area. At low water the pools have much the larger section, at high water the bars. Let Figs. 18 and 19 represent two such sections. At a medium stage the two will have about an equal content, say, 180 000 sq. ft. Now let the depth be diminished by 10 ft. The width of the broad section is twice as great as of the narrow. It will therefore lose, perhaps, 65 000 sq. ft. of area, while the narrow section loses only 32 500 sq. ft. Will a given decrease of discharge produce a fall of equal amount at the two sections, widely diverse in form as they are? Experience shows that at two stations, only a few miles apart, the rate of rise or fall is nearly uniform, irrespective of the shape of the section. Reasons will be given hereafter why this must be so.

Now the discharge that passes any station does not depend in the least upon the slope, depth or condition of the bed at that place. It depends solely upon the quantity of water furnished from above. While the supply continues the same, the discharge is constant. If, then, the area be increased, the velocity must be decreased. Either this, or the water surface must suffer a fall sufficient to compensate the increase of area. Experience shows that no such fall occurs, and reason asserts that it could not occur. On January 9th and 10th, 1885, at Arkansas City, a scour of 11 237 ft. is recorded. The width of the river was 3 370 ft. If the same velocity had been maintained after the change of area as had existed before it, a fall would have occurred of 3.3 ft. Instead, the rise which was steadily progressing at the rate of 0.5 ft. per day was only slightly disturbed. Indeed, in a stretch of river which has a mean fall of only 0.32 ft. to the mile, it is evident that no serious change of level can take place without disturbing the whole stream. That there was a slight change of level is evident from an inspection of the slope curve in Fig. 16, where a palpable disturbance is shown at about 38 ft. The change was only 0.1 or 0.2 ft. This is a striking proof of the reality of the fluctuations of area.

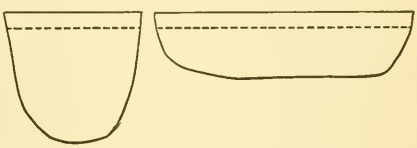


FIG. 18. FIG. 19.

It is not possible, however, that the velocity should diminish while the slope remains unaltered and the depth increases. The local slope must therefore accommodate itself to the change of depth and velocity. A change of slope of 0.048 ft. to the mile would compensate for

an increase in depth of 3.3 ft. and a decrease of velocity of 0.3. This change, of course, is maintained over only a short distance.

So long as the cross-sections are equal, the velocities will be equal also. As soon as the wide cross-section becomes less than the narrow, the velocity of the former will become greater than that of the latter, and this disparity will increase as long as the downward movement of the river continues. If the bed of the river were unalterable, the ultimate result would be that the current over the bars would approximate to the condition of a cascade or rapid, while that in the pools would be comparatively stagnant.

The bed of the river, however, is far from unalterable; the effect of the increased velocity over the bars in time of low water is to produce a scour, while that of the diminished velocity in the pools is to promote a deposit of the suspended matter brought down from the bar above. During a rise, the cross-section over the bar increases more rapidly than that in the pool and soon surpasses it in area. Its velocity then diminishes as compared with that of the narrow section. Soon the current in the wide section becomes slack, while that of the pool keeps increasing in rapidity. The latter then becomes the scene of scour, while at the former a fill is taking place. Hence, the well known phenomenon, which is regularly repeated every season, of the transference of enormous quantities of material from place to place with every great change of stage. These movements are of great magnitude, the datum cross-section nearly always undergoing alterations in a single season of 25 000 to 30 000 sq. ft.*

Secular Changes of Area.—The secular or permanent changes of area have already been alluded to, and Greenville and Wilson's Point have been cited as examples of what seems a permanent enlargement of section. So far as Greenville is concerned, this fact cannot be established by direct measurement, for no discharge observations have been taken there. At Wilson's Point there is satisfactory evidence that the cross-section has increased in area since 1890 by about 30 000 sq. ft. The greater part of this enlargement of area has taken place above the low-water line, as Captain Townsend has remarked.† Whereas the whole section has experienced the augmentation mentioned above, the low-water area seems to have increased only about 5 000 sq. ft.

* These points are well elucidated in a paper "On the Protection of the Lower Mississippi Valley from Overflow," by J. B. Johnson, M. Am. Soc. C. E., in the *Journal* of the Association of Engineering Societies, Vol. III, No. 9.

† "Report of Mississippi River Commission for 1893," p. 3762.

It would, perhaps, be possible to trace out the changes of the river-bed in other sections by a careful study of the slopes. The data for such a study are abundant, and are valuable for their continuity and their accuracy. They have been accumulating for 24 years; they are certain and exact within very narrow limits. There is no room for any considerable error in them. Such a study, however, would be laborious, and the conclusions would have to be freed carefully from perturbations due to other causes than change of area. It is hardly likely that it will be undertaken.

It is highly probable that a series of years of unusually high or unusually low water will leave its impress on the bed of the river. In fact, there is good evidence that such is the case.

Discussion of the Discharge Observations of 1884-85 at Arkansas City.

With this preparation for the investigation, an attempt will be made to develop some law of coefficients of the ordinary formula, using the general slope and the mean depth. For this purpose the discharge observations made at Arkansas City in 1884-85 will be used.* This place is selected because it is situated at the head of a reach wholly confined by levees, and below the confluence of the two greatest tributaries of the lower Mississippi; and because there is a station 93 miles below, Wilson's Point, where discharges have also been taken for several years, thus affording a valuable check on the measurements made at the upper station. The observations of 1884-85 are the first ever taken at Arkansas City. In the discussion by Mr. Wheeler, several sources of error are pointed out. Prior to October 12th observations were made with the meter at not more than 12 ft. below the surface. No corrections have been applied to reduce them to mean velocity. It will be seen that the coefficients deduced from them are much too great. Subsequently to October 12th velocities were taken at 0.6 of the depth, and the results accepted as the mean velocities in those verticals. There is room for error in the areas, owing to large changes in the lead-line, but, as it is unknown where they occurred, it is useless to attempt to correct them.

Table No. IV contains the principal data of the discharge observations of 1884-85, together with the slope, as estimated from the gauge records, from Arkansas City to Greenville, the theoretical velocity

* "Report of the Mississippi River Commission for 1887," pp. 2835-40.

TABLE No. IV.—Discharge Observations at Arkansas City, Ark.

Date.	Gauge reading.	Dimensions of Cross-Section of Discharge.					Mean velocity per second.	Discharge per second.	Slope, Arkansas City-Greenville.	Theoretical velocity.	Coefficient.	Direction and force of wind.
		Area.		Mean depth.	Width.	Scour or fill.						
		Water.	Below datum.									
1884.	Feet.	Sq. ft.	Sq. ft.	Feet.	Feet.	Sq. ft.	Feet.	Cu. ft.				
October 1	7.80	.....	.....	.....	.....	.....	.....	.....	0.00003708	.....	.....	
2	8.70	.....	.....	.....	.....	.....	.....	.....	0.00003897	.....	.....	
3	10.40	.....	.....	.....	.....	.....	.....	.....	0.00004217	.....	.....	
4	12.50	.....	.....	.....	.....	.....	.....	.....	0.00004539	.....	.....	
5	14.80	.....	.....	.....	.....	.....	.....	.....	0.00004791	.....	.....	
6*	16.60	139 391	223 301	42.5	3 282	.....	3.263	454 778	0.00004876	4.55	71.7	
7*	18.05	145 106	224 247	44.2	3 285	+ 946	3.344	485 275	0.00004937	4.67	71.6	
8	19.25	153 954	229 149	46.7	3 298	+4 902	3.472	534 514	0.00005042	4.85	71.6	
9*	20.00	149 885	222 606	45.4	3 302	−6 543	3.594	538 721	0.00005042	4.78	75.1	
10*	20.60	159 929	230 664	48.4	3 302	+8 058	3.449	551 566	0.00005009	4.92	70.1	
11	21.00	.....	.....	.....	.....	.....	.....	.....	0.00005042	.....	.....	
12	20.65	.....	.....	.....	.....	.....	.....	.....	0.00004923	.....	.....	
13	20.15	153 144	225 370	46.4	3 299	−5 294	3.149	482 228	0.00004862	4.75	66.3	
14	19.50	150 089	224 469	45.5	3 299	— 910	2.977	446 854	0.00004820	4.68	63.6	W.
15	18.85	145 259	221 773	44.1	3 295	−2 088	2.915	428 418	0.00004796	4.60	63.4	Calm.
16	18.40	143 698	221 695	43.6	3 295	— 77	2.826	406 063	0.00004791	4.57	61.8	Do.
17	18.15	143 583	222 404	43.6	3 295	+ 709	2 972	426 741	0.00004749	4.55	65.3	
18	18.10	144 477	223 463	43.8	3 295	+1 059	2,863	413 667	0.00004749	4.56	62.8	Do.
19	18.25	.....	.....	.....	.....	.....	.....	.....	0.00004782	.....	.....	
20	18.47	144 501	222 268	43.8	3 296	−1 195	2,806	405 416	0.00004744	4.58	61.2	N.E., light.
21	18.50	144 034	221 702	43.7	3 296	— 566	2.907	418 640	0.00004712	4.54	64.1	S.E., light.
22	18.42	143 372	221 304	43.5	3 295	— 398	2,890	414 295	0.00004744	4.54	63.6	
23	18.20	141 900	220 556	43.1	3 295	— 748	2.943	417 592	0.00004726	4.51	65.2	W., strong.
24	17.95	139 656	219 137	42.4	3 295	−1 419	2.823	394 222	0.00004721	4.47	63.1	E., light.
25	17.55	138 468	219 257	42.1	3 286	+ 120	2.710	375 263	0.00004688	4.44	61.0	
26	17.10	.....	.....	.....	.....	.....	.....	.....	0.00004650	.....	.....	
27	16.75	135 446	218 861	41.3	3 281	— 396	2,773	375 631	0.00004580	4.35	64.0	Calm.
28	16.43	133 525	217 990	40.7	3 279	— 871	2,676	357 325	0.00004547	4.30	62.2	W., strong,
29	16.20	131 501	216 718	40.1	3 277	−1 272	2,691	353 917	0.00004571	4.28	62.9	Calm.

	30	16.05	130 136	215 847	39.7	3 277	— 871	2.707	352 295	0.00004594	4.27	63.4	Do.
	31	16.30	132 444	217 334	40.4	3 278	+1 487	2.660	352 249	0.00004608	4.31	61.7	Do.
November	1	16.50	134 536	218 775	41.0	3 282	+1 441	2.723	366 394	0.00004641	4.36	62.4	Do.
	2	16.60								0.00004641			
	3	16.40	131 074	215 643	39.9	3 282	—3 132	2.688	352 363	0.00004608	4.29	62.7	Do.
	4†	16.25	130 370	215 430	39.7	3 281	— 213	2.750	358 517	0.00004594	4.27	64.4	N. W., strong.
	5	16.10	130 578	216 126	39.8	3 278	+ 696	2.774	362 230	0.00004580	4.27	64.9	N. W., light.
	6	16 00	130 875	216 750	39.9	3 277	+ 624	2 741	358 775	0.00004571	4.27	64.2	W.
	7	15.70	129 375	216 232	39.5	3 275	— 518	2.669	345 291	0.00004547	4.24	63.0	Calm.
	8	15.20	126 492	214 931	38.7	3 270	—1 251	2 673	338 146	0.00004509	4.17	64.1	Do.
	9	14.65								0.09004406			
	10	14.00	121 535	213 951	37.2	3 265	—1 030	2.575	312 907	0.00004311	4.00	64.3	Do.
	11	13.35	120 405	214 938	36.9	3 260	+ 987	2.486	299 275	0.00004297	3.98	62.4	Do.
	12	12.75	118 694	215 177	36.5	3 255	+ 239	2.496	296 235	0.00004241	3.93	63.4	Do.
	13	12.25	115 843	213 949	35.6	3 251	—1 228	2.460	284 955	0.00004123	3.83	64.2	Do.
	14	11.70	113 466	213 361	34.9	3 248	— 588	2.370	268 895	0.00001100	3.78	62.6	Do.
	15	11.18	112 829	214 413	34.8	3 245	+1 052	2.377	268 251	0.00004085	3.77	63.0	Do.
	16	10.70								0.00004029			
	17	10.25	108 334	212 859	33.6	3 224	—1 554	2.349	254 452	0.00003991	3.66	64.1	Do.
	18	9.85	107 053	212 851	33.8	3 165	— 8	2.270	243 034	0.00003959	3.66	62.1	N. W., light.
	19	9.50	105 972	212 881	33.6	3 154	+ 30	2 472	261 952	0.00003959	3.65	67.8	Do.
	20	9.25	104 764	212 462	33.6	3 119	— 419	2.298	240 774	0.00003865	3.60	63.8	Calm.
	21	9.05	103 376	211 708	33.1	3 119	— 754	2.419	250 087	0.00003921	3.60	67.1	S. E., light.
	22	8.93	103 916	212 628	33.3	3 119	+ 920	2.435	253 004	0.00003897	3.60	67.6	E., strong.
	23	9.00								0.00003912			
	24	8.90	103 042	211 849	33.0	3 119	— 779	2.521	259 781	0.00003902	3.59	70.4	Calm.
	25	8.83	103 046	212 075	33.0	3 119	+ 226	2.357	242 859	0.00003865	3.57	66.0	S. W., light.
	26	8 67	102 466	212 002	32.9	3 118	— 73	2.399	245 850	0.00003865	3.56	67.3	Do.
	27	8.58	101 707	211 526	32.7	3 115	— 476	2.342	238 191	0.00003865	3.55	65.9	Do.
	28	8.50								0.00003817			
	29	8.66	101 842	211 407	32.7	3 115	— 119	2.362	240 522	0.00003832	3.54	66.7	
	30	9.10								0.00003973			
December	1	9.77	105 412	211 466	33.4	3 154	+ 59	2.454	268 684	0.00004048	3.68	66.8	S., light.
	2	10.66	107 216	210 427	32.2	3 229	—1 039	2.637	282 728	0.00004161	3.66	72.0	Calm.
	3	11.38	110 268	211 198	34.0	3 245	+ 771	2.609	287 696	0.00004170	3.76	69.3	Do.
	4	11.89	111 773	211 046	34.4	3 248	— 152	2.617	292 500	0.00004320	3.85	67.9	S., strong.
	5	12.00	111 208	210 129	34.2	3 250	— 917	2.581	287 037	0.00004241	3.81	67.8	S. E., strong.
	6*	11.60	110 719	210 942	34.1	3 248	+ 813	2.894	320 386	0.00004161	3.77	77.3	S., very strong.
	7	10.85								0.00003996			
	8	10.32	106 816	211 137	33.1	3 229	+ 195	2.434	259 964	0.00003921	3.60	67.4	Do.
	9	9.55	103 542	210 297	32.8	3 154	— 840	2.431	251 696	0.00003874	3.56	68.2	Do.
	10	8.80	100 444	209 528	32.2	1 119	— 769	2 465	247 521	0.00003827	3.50	70.2	Calm.
	11*	8.22	98 290	209 249	31.6	3,115	— 279	2.626	258 131	0.00003779	3.47	74.2	S., strong.

* Reject.

† Water rough.

TABLE No. IV—Discharge Observations at Arkansas City, Ark.—(*Continued*).

Date.	Gauge reading.	Dimensions of Cross-Section of Discharge.					Mean velocity per second.	Discharge per second.	Slope, Arkansas City-Greenville.	Theoretical velocity.	Coefficient.	Direction and force of wind.
		Area.		Mean depth.	Width.	Scour or fill.						
		Water.	Below datum.									
1884.	Feet.	Sq. ft.	Sq. ft.	Feet.	Feet.	Sq. ft.	Feet.	Cu. ft.				
December 12	7.82	96 848	209 073	31.1	3 114	— 176	2.418	234 212	0.00003732	3.41	71.0	W., strong.
13	7.42	94 694	208 202	30.4	3 111	— 871	2.338	221 390	0.00003746	3.37	69.3	N. E., light.
14	7.50	.....	.....	.....	.....	.....	.....	.....	0.00003784	.....	.....	
15	8.84	99 689	208 684	32.0	3 115	+ 482	2.438	242 997	0.00004100	3.62	67.3	Calm.
16	10.94	107 009	209 306	33.1	3 229	+ 622	2 676	286 283	0.00004453	3.75	69.7	S. E., light.
17	12.76	112 401	208 851	34.5	3 255	— 455	2.853	320 685	0.00004632	4.00	71.4	N. W., strong.
18*	15.01	116 979	306 087	35.8	3 268	—2 764	2.883	337 316	0.00004688	4.10	70.5	Do.
19	16.57	124 801	208 810	38.0	3 282	+2 723	2.925	365 023	0.00004806	4.27	68.4	N., light.
20	17.58	127 399	208 089	38.8	3 286	— 721	3.059	389 672	0.00004806	4.32	70.8	S. S., light.
21	18.10	.....	.....	.....	.....	.....	.....	.....	0.00004782	.....	.....	
22	18.41	131 855	209 820	40.0	3 296	+1 731	3.144	414 584	0.00004806	4.38	71.7	N., strong.
23	18.41	128 318	206 283	38.9	3 296	—3 537	3.060	392 724	0.00004829	4.33	70.6	N., light.
24†	18.14	128 980	207 834	39.1	3 295	+1 551	3.076	396 752	0.00004759	4.32	71.2	N., strong.
25	17.85	.....	.....	.....	.....	.....	.....	.....	0.00004749	.....	.....	
26	16.80	123 438	206 689	37.6	3 281	—1 145	2.957	365 000	0.00004658	4.18	70.7	Calm.
27	15.85	.....	.....	.....	.....	.....	.....	.....	0.00004477	.....	.....	
28	15.35	.....	.....	.....	.....	.....	.....	.....	0.00004571	.....	.....	
29	14.98	117 421	206 633	35.9	3 270	— 56	2.895	339 965	0.00004905	4.19	68.9	E., strong.
30	16.85	121 488	204 574	37.0	3 281	—2 059	3.093	375 724	0.00004759	4.20	73.7	N., strong.
31	20.28	131 365	203 150	39.8	3 299	—1 424	3 486	457 871	0.00005168	4.53	76.9	N. W., strong.
1885.												
January 1†	23.33	142 439	204 104	43.1	3 305	+ 954	3.913	557 296	0.00005325	4.79	81.6	N. W., very strong.
2	26.13	154 537	206 901	46.6	3 313	+2 797	4.123	637 141	0.00005371	5.00	82.4	N., light.
3	28.71	162 507	206 285	48.8	3 328	— 616	4.428	719 521	0.00005433	5.15	86.0	Calm.
4	31.10	.....	.....	.....	.....	.....	.....	.....	0.00005528	.....	.....	
5	32.90	.....	.....	.....	.....	.....	.....	.....	0.00005499	.....	.....	
6	34.55	.....	.....	.....	.....	.....	.....	.....	0.00005645	.....	.....	
7	35.55	184 498	205 322	54.9	3 359	— 963	4.837	892 383	0.00005688	5.59	86.6	S., light.
8	36.30	.....	.....	.....	.....	.....	.....	.....	0.00005726	.....	.....	

	9	36.85	194 569	211 039	57.7	3 370	+5 717	4.982	969 430	0.00005740	5.75	86.6	N., light.
	10	37.30	201 608	216 559	59.8	3 370	+5 520	4.863	980 502	0.00005773	5.87	82.8	E., light.
	11	37.60								0.00005670			
	12	38.10	207 121	219 374	61.4	3 371	+2 815	4.907	1 016 372	0.00005679	5.90	83.1	N., light.
	13	38.49	202 053	212 990	59.9	3 371	−6 384	4.798	969 431	0.00005735	5.86	81.9	N., strong.
	14	39.03	202 334	211 448	60.0	3 371	−1 542	5.167	1 045 446	0.00005773	5.89	87.8	N. E., light.
	15	39.80								0.00005858			
	16†	40.28	207 854	212 748	61.7	3 371	+1 300	5,374	1 117 022	0.00005867	6.02	89.3	N. W., strong.
	17	40 61	207 350	211 131	61 5	3 372	−1 617	5 152	1 068 270	0.00005867	6.01	85.8	W., strong.
	18	40.95								0.00005927			
	19	41.04	207 404	209 734	61.5	3 374	−1 397	5,307	1 100 708	0.00005923	6.04	87.0	N., light.
	20	41.20	209 210	211 000	62.6	3 375	+1 266	5,114	1 069 814	0.00005914	6.08	84.1	N. W., light.
	21	41.30								0.00005923			
	22	41.35	207 877	209 161	61.6	3 376	−1 839	5,158	1 072 266	0.00005891	6.02	85.6	N., light.
	23	41.48	211 451	212 296	62.6	3 377	+3 135	5.165	1 092 097	0.00005891	6.07	85.1	Calm.
	24	41.50								0.00005900			
	25	41.50								0.00005853			
	26	41.50								0.00005867			
	27	41.60								0.00005914			
	28	41.70								0.00005952			
	29	41.70								0.00005914			
	30	41.69	218 608	218 748	64.7	3 380	+6 452			0.00005891	6.17		
	31	41.73	217 778	217 778	64.4	3 380	− 970	4,888	1 064 462	0.00005867	6.15	79.5	W., light.
February	1	41.70								0.00005867			
	2	41.61	216 234	216 640	64.0	3 380	−1 138	4 979	1 076 725	0.00005881	6.13	81.1	S., light.
	3	41.34	216 288	217 606	64.0	3 380	+ 966	4.853	1 049 734	0.00005797	6.09	79.7	Do.
	4	40.79	217 134	220 308	64.4	3 374	+2 702	4.727	1 026 309	0.00005773	6.10	77.5	S., strong.
	5	39.90	215 970	222 147	64.1	3 371	+1 839	4,541	980 803	0.00005561	5.97	76.1	W., very strong.
	6	38.72	212 233	222 393	63.0	3 371	+ 246	4.382	929 904	0.00005628	5.90	74.3	S., light.
	7	37.46	205 316	219 729	60.9	3 371	−2 664	4.229	868 374	0.00005410	5.74	73.6	S., strong.
	8	36.30								0.00005324			
	9	35.14	193 968	216 189	57.7	3 364	−3 540	4.058	787,120	0.00005301	5.53	73.4	W., strong.
	10	34.15								0.00005277			
	11	33.59	188 421	215 837	56.2	3 356	− 352	3.944	743 225	0.00005395	5.51	71.6	S., light.
	12	33.19	184 158	212 912	54.9	3 354	−2 925	3,966	730 365	0.00005386	5.44	72.9	N.
	13	32.93	184 790	214 420	55.1	3 354	+1 508	4 074	752 836	0.00005409	5.46	74.6	N., light.
	14	32.70	184 080	214 484	54.9	3 354	+ 64	4.171	767 829	0.00005409	5.45	76.9	Calm.
	15	32.55								0.00005442			
	16	32.25	179 871	211 785	53.6	3 353	−2 699	4,078	733 525	0.00005456	5.41	75.4	W., light.
	17	32.18	181 758	213 908	54.2	3 353	+2 123	4,178	759 442	0 00005433	5.43	77.0	S., light.
	18	32.25	179 674	211 588	53.6	3 353	−2 320	4.090	734 787	0.00005442	5.40	75.7	W., strong.
	19	32.30								0.00005451			
	20	32.20	180 956	213 039	54.0	3 353	+1 451	4.188	757 822	0.00005433	5.42	77.3	N., strong.

* Reject.

† High waves.

TABLE No. IV.—Discharge Observations at Arkansas City, Ark.—(*Continued*).

Date.	Gauge reading.	Dimensions of Cross-Section of Discharge.					Mean velocity per second.	Discharge per second.	Slope, Arkansas City-Greenville.	Theoretical velocity.	Coefficent.	Direction and force of wind.
		Area.		Mean depth.	Width.	Scour or fill.						
		Water.	Below datum.									
1895.	Feet.	Sq. ft.	Sq. ft.	Feet	Feet.	Sq. ft.	Feet.	Cu. ft.				
February 21	31.97	180 207	213 064	53.7	3 353	+ 25	4.028	725 963	0.00005442	5.41	74.5	N. E., light
22	31.35	.....	.....	.....	.....	.....	.....	.....	0 00005334	.....	.....	.....
23	30.52	175 006	212 694	52.3	3 344	— 370	3.822	668 902	0.00005301	5.27	72.6	N. E., strong.
24	29.64	168 823	209 499	50.5	3 342	—3 195	3.707	625 866	0.00005254	5.15	72.0	N. W., light.
25	28.45	162 084	206 738	48.6	3 338	—2 761	.....	....	0.00005221	5.04	.....	Calm.
26	27.05	.....	.....	.....	.....	.....	.....	.....	0.00005004	.....	.....	.....
27	25.39	152 179	207 050	45.7	3 331	+ 312	3.354	510 424	0.00005004	4.78	70.1	S , light.
28	24.01	146 272	205 744	43.9	3 328	—1 306	3.311	484 270	0.00004876	4.63	70.6	E., light.
March 1	22.80	.....	.....	.....	.....	.....	.....	.....	0.00004867	.....	.....	.....
2	22.09	144 366	210 146	43.7	3 302	+4 402	3 255	469 891	0.00004914	4.63	70.1	Calm.
3	21.81	144 518	211 225	43.8	3 301	—1 079	3.166	457 584	0.00004970	4.67	67.9	Do.
4	21.77	145 777	212 617	44.2	3 301	+1 392	3.266	476 135	0.00005027	4.71	69.3	N. E., strong.
5	21.85	144 162	210 736	43.7	3 301	—1 881	3.182	458 682	0.00005032	4.69	67.9	N., strong.
6	22.15	145 437	211 013	44.1	3 301	+ 277	3.237	470 825	0.00005065	4.73	68.5	S. W., light.
7	22.62	146 945	210 962	44.5	3 302	— 51	3.340	490 803	0.00005159	4.79	69.7	N., strong.
8	23.15	.....	.....	.....	.....	.....	.....	.....	0.00005207	.....	.....	.....
9	23.83	152 241	212 298	45.8	3 323	+1 336	3.309	503 789	0.00005278	4.92	67.3	S. E., light.
10	24 53	154 370	212 111	46.4	3 330	— 187	3,460	534 116	0.00005362	4.99	69.4	S. W., strong.
11	25.48	155 650	210 222	46.7	3 332	—1 889	3.486	542 651	0.00005381	5.02	69.5	S W., light.
12	26.70	158 835	209 340	47.6	3 337	— 882	3,576	567 990	0.00005490	5.11	70.0	S., light.
13	28.08	164 755	210 648	49.3	3 340	+1 308	3.827	630 445	0.00005561	5.24	73.1	W., light.
14	29.26	168 436	210 385	50.3	3 346	— 263	3.747	631 086	0.00005584	5.30	70.7	S., light.
15	30.20	.....	.....	.....	.....	.....	.....	.....	0.00005575	.....	.....	.....
16	30.87	174 438	210 998	52.0	3 353	+ 613	4.002	698 138	0.00005551	5.37	74.5	S., light.
17	31.37	174 567	209 454	52.0	3 355	—1 544	4.107	716 933	0.00005537	5.36	76.5	N., strong.
18	32.02	180 981	213 699	53.9	3 359	+4 245	4.100	741 980	0.00005593	5.49	74.7	N. W., light.
19	32.71	178 824	209 230	53.2	3 362	—4 469	4.287	766 661	0.00005622	5.47	78.4	W., strong.
20	33.41	180 488	208 543	53 6	3 364	— 687	4.316	779 062	0.00095688	5.52	78.2	N., light,
21	34.01	184 859	210 902	54.9	3 367	+2 359	4.186	773 810	0.00005679	5.58	75.0	

	22	34.40								0.00005608			
	23	34.75	137 655	211 209	55.7	3 369	+ 307	4.202	788 488	0.00005584	5.58	75.4	N., light.
	24	34.83	192 356	215 643	57.1	3 370	+4 437	4.165	801 188	0.00005608	5.66	73.6	N. E., light.
	25	34.86	188 825	212 011	56.0	3 370	−3 632	4 193	791 786	0.00005670	5.63	74.4	Do.
	26	34.75	189 316	212 873	56.2	3 370	+ 862	4.234	801 535	0.00005617	5.62	75.4	Calm.
	27	34.55	188 810	213 039	56.0	3 369	+ 166	4.181	789 380	0.00005546	5.57	75.0	S., light.
	28	34.28	188 165	213 290	55.9	3 365	+ 251	4.352	818 810	0.00005504	5.55	78.5	N. W., strong.
	29	33.90								0.00005499			
	30	33.30	185 640	214 058	55.2	3 362	+ 768	4.002	742 849	0.00005433	5.48	73.1	S., light.
	31	32.60	181 149	211 908	53.9	3 358	−2 150	3.946	714 826	0.00005395	5.39	73.2	Calm.
April	1	31.75	178 475	212 078	53.2	3 354	+ 170	3.851	687 259	0.00005419	5.37	71.7	S. E., light.
	2	30.80	173 935	210 714	51.9	3 350	−1 364	3.844	668 617	0.00005301	5.25	73.3	S., strong.
	3	29.90	167 984	207 790	50.1	3 351	−2 924	3.896	654 455	0.00005240	5.12	76.1	N. W., strong.
	4	28.86	165 699	208 986	49.5	3 345	+1 196	3.729	617 823	0.00005254	5.10	73.1	N., light.
	5	28.00								0.00005183			
	6	27.34	159 934	208 303	47.9	3 340	− 683	3.512	561 739	0.00005159	4.97	70.7	S., light.
	7	26.92	157 807	207 579	47.3	3 339	− 724	3.553	560 652	0.00005159	4.94	71.9	S. E., light.
	8	26.77	157 628	207 908	47.2	3 343	+ 329	3.544	558 663	0.00005254	4.98	71.2	E., strong,
	9	27.06	157 554	206 864	47.1	3 343	−1 044	3.601	567 279	0.00005301	5.00	72.1	S. E., light.

NOTE.—Most of the computations of theoretical velocities, etc., in this paper were made with the slide rule. Hence there may frequently be errors in the final figures.

according to the formula $v = 100\sqrt{rs}$, and the coefficients required to bring the expression $\sqrt{rs}$ to the velocity as observed.

Curve of Coefficients.—The principal object of the investigation is to arrive at the relation between discharge and gauge height. It is evident from a casual inspection of the table that the coefficients, on the whole, increase as the river rises and diminish as it falls. To develop the law of this change, take the gauge as an argument and

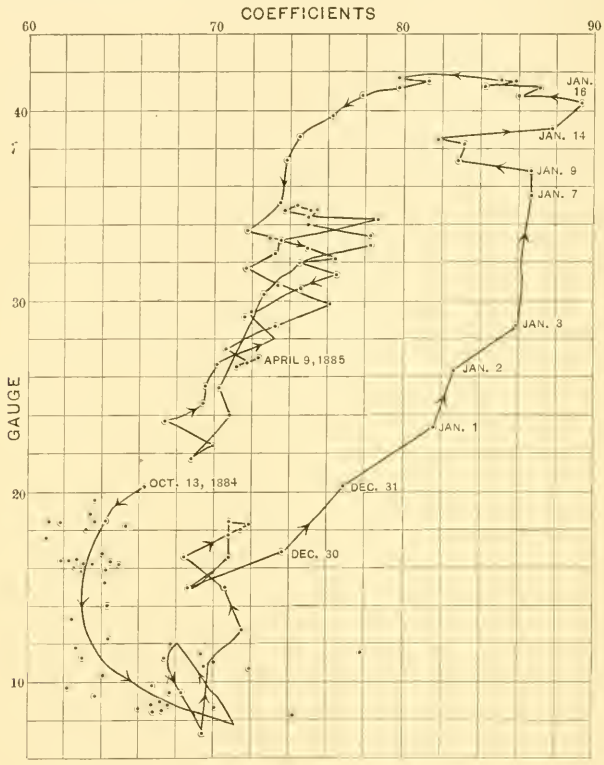


FIG. 20.

plot the coefficients as functions, as in Fig. 20. It will be seen that they do not by any means follow a regular law of progression with the gauge, but that the curve which they describe is an irregular oval, the greater values of the coefficients corresponding, on the whole, to the ascending branch, representing a rising river, and the less values to a falling river. This general law is subject, however, to some

remarkable exceptions, there being violent departures from the curve, sometimes followed by as sudden returns. If the table of data is examined, it will be found that these, for the most part, as might have been expected, are contemporaneous with changes of area in the opposite direction. A further examination will reveal the fact that the two branches of the curve themselves, the ascending and the descending, also represent less and greater areas respectively.

Arkansas City has a narrow and deep section; in short, the river there is a pool, and in accordance with the general law the cross-section undergoes enlargement by scour with a rise, and contraction by fill with a fall. Now, if the controlling section of a river, whatever that be, remained unaltered, every change in the area at Arkansas City would be exactly compensated by an inverse change of velocity. The discharge, indeed, would not always be the same for the same gauge height, for the velocity would still be affected by the changes of slope due to rises and falls; but the coefficients, which take cognizance of the changes of slope, would follow a regular law, subject to small perturbations.

On the other hand, a persistence of datum area at any station, while the adjacent reaches, above or below or both, are undergoing mutation, will produce a change of velocity at the station, even though the general slope and mean depth remain the same; or if the latter elements also vary, as they usually do, then the change of velocity will be out of all proportion to the variation in those elements; or, in other words, the coefficient will show a marked departure from its ordinary rate of increase or decrease. The same effect will follow in a less degree if the areas do not remain absolutely persistent, but fail to respond at once to the altered conditions elsewhere. A very striking exemplification of this statement is afforded by the high-water discharge of 1890 at Wilson's Point, of which a plotting is given in Fig. 12, previously referred to. It will be seen that the coefficient varied, from January 28th to March 13th, from 146 to 97, the river steadily rising. The movement of the coefficient, on the whole, was tolerably regular. Observing now the datum areas, it will be found that they increased steadily from about 159 000 to 193 000 during the same time, and remained at about that figure till the close of the observations. The inference is irresistible that here is the true explanation of this phenomenon; and it shows the futility of attempting to apply formulas

based on general conditions to the particular circumstances existing at any one place, without a knowledge of those circumstances and a careful scrutiny of the disturbing causes which they introduce.

It is not necessary to assume any alteration in the controlling section to account for the increased velocity at the discharge station. The latter, as at Arkansas City, and as is generally the case, is a narrow section, much less than the average of the river, and the rapid rise had poured down a flood, which increased faster than the contracted section at Wilson's Point could accommodate it at the usual velocity. Hence the latter had to be increased, which event happened, no doubt, by steepening the local slope, though the movement was attended by so slight a disturbance of the water level that it made little or no impression on the general slope. The velocity of 7 ft. per second or more that was attained rapidly produced a scour, and, as the area enlarged, the velocity fell by degrees to the normal.

An equally significant example of the fallacious indications afforded by strictly local conditions is presented by the Helena observations of 1884-85. At this point there were two sections, one for high and the other for low water, about 0.75 mile apart. On two days measurements were taken at both. The general slope, of course, was the same for both sections. Yet at the upper (low water) station, on a given date, the mean depth was 19.7, the velocity 3.06, and the coefficient 71, while at the lower the mean depth was 14.1, the velocity 3.82, and the coefficient 105. On a subsequent date the velocities were 2.85 and 3.59, the coefficients 71 and 95. On the first day the discharges measured at the two sections differed by 25 000 cu. ft.; on the second they were almost identical in value.

Formulas Give Too Low Velocities at Low Stages.—It may here be remarked that the formulas all give too low values for the velocity at low stages, and consequently too high values for the coefficients, and those formulas which, like Weisbach's and Kutter's, give especial prominence to the depth, err particularly in this regard. The reason for this error is that they take the general mean depth of the section, obtained by dividing the area by the wetted perimeter, instead of dividing the section into partial areas and applying the formula to each, an error identical in principle with that mentioned on page 54, whereby the mean velocity is confounded with the mean of the mean velocities in vertical planes. The amount of error thus introduced

will vary with the shape of the section and the stage of water, being greatest when the section is composed of a deep and a shallow part, and when the latter is first submerged. In sections of uniform depth it is nothing Generally it is greatest at low stages, the variations of depth being relatively less when the river attains its height. In the present instance, at the lowest stage, 7.42 ft. on the gauge, the error is about 10%; at 20 ft. it is about 8%; at the bank-full stage it is about 4 per cent.

Fill at Low Stage.—An inspection of Table IV will show that the fill progresses very steadily at low stages, the area decreasing from 230 000 to 203 000. It may be a matter of surprise that at so low a stage, when the river has so little transporting power, the movement of material should be so considerable and so persistent, when at much higher velocities, as from December 30th to January 23d, it should be comparatively insignificant. It should be remarked, however, that the material which is deposited on the bottom is not brought from afar, but is, in all probability, derived from the bar immediately above, and that the velocity prevailing on that bar is abundantly sufficient to dislodge and transport considerable quantities of sediment, the deposition of which is favored by the low velocity in the pool. It is probable also that a good deal of matter is rolled along the bottom.

The relation between transporting power and velocity is not definitely known, and there is some diversity in the views of engineers on the subject. It is generally admitted, however, that there is a connection between the two, and that an increase of velocity is attended by a scour, and a diminution of velocity by a deposit of the matter already in suspension.

Supposition of Uniform Velocity.—Now suppose a stream, straight in plan and of erodable bed, to be actuated by a current which is perfectly uniform. If the water enter the head of this stream already loaded with all that it can carry at that velocity, it will take up no more, but will transport its burden without accession or loss to the sea. If the course have bends in it, the force of the current will be exerted mechanically against the banks presented at abrupt angles, and an additional load will be temporarily taken up to be deposited presently at the reversion points of the bends. The general result will be the same, that a load will be delivered at the mouth equal in quantity to that carried at the head of the alluvial basin. This, of course, on

the supposition that the material carried is homogeneous with that of the bed and banks.

There will still be a quantity of matter rolled along by the current. According to the received ideas, there is always velocity enough prevailing in the Mississippi to move the loose and finely divided material which lines its channel. From the well-worn tables of Dubuat it would appear that a mean velocity of less than 1 ft. per second will move sand and even coarser substances. Both Mr. Powless and Mr. Hider found the movements of sand waves to proceed at low water as well as at high. The movements, however, were more violent and the mass of material moved was greater as the velocity increased.

Scour or Fill Depends on Differences of Velocity.—It will therefore be seen that the scour or fill at any station does not depend on the absolute velocity at that place, but on the difference of velocity between it and the adjacent station. If the velocity were the same, no matter how great, the same quantity would leave the section as entered it. While, therefore, there might be a very large movement of material through the section, measurements would not show any permanent changes in its dimensions. There might be considerable daily disturbances, due to the passage of sand waves.

The alternation of wide and narrow places is very well exemplified at Arkansas City. About 3 miles above the discharge section, which is about 3 400 ft. wide, is a wide and shallow stretch about 7 750 ft. broad. According to the soundings of 1882, at a stage of 42.4 on the Arkansas City gauge, the area of this section was about 233 000 sq. ft., while that of the narrow section just below the town was 203 700. If the areas are assumed to remain unaltered by fill or scour, there will be found at 35 ft., areas of 178 300 and 178 400; at 30 ft., 139 700 and 161 400; at 20 ft., 119 800 and 128 500; at 7.4 ft. (low water of 1884–85) 51 600 and 86 500. Fig. 21 gives plots of the two sections. It will be observed that at the extremely low stage, the wide place has contracted its breadth to hardly more than half of its former proportions, and has become almost a pool itself. These conditions, of course, do not continue, for the small cross-section enlarges and the great one contracts, at either stage, by the action of the river.

There is a time during the rise and also during the fall of every flood when the velocities in the deep and in the shallow places approach equality. At such times there should be little general change in the dimen-

sions of the bed. At Arkansas City, in 1882, assuming the bed to remain as it was when the soundings were made for the general map, at a stage of about 35 ft. on the Arkansas City gauge, the area in the two sections was about the same, namely, about 178 400 sq. ft., and the velocity about 4.66. Under such circumstances the changes in the dimensions of the sections should be principally from material rolled along the bottom.

The tendency, then, at each stage, is toward equalization of the areas of the wide and narrow sections. As soon as the area, either on

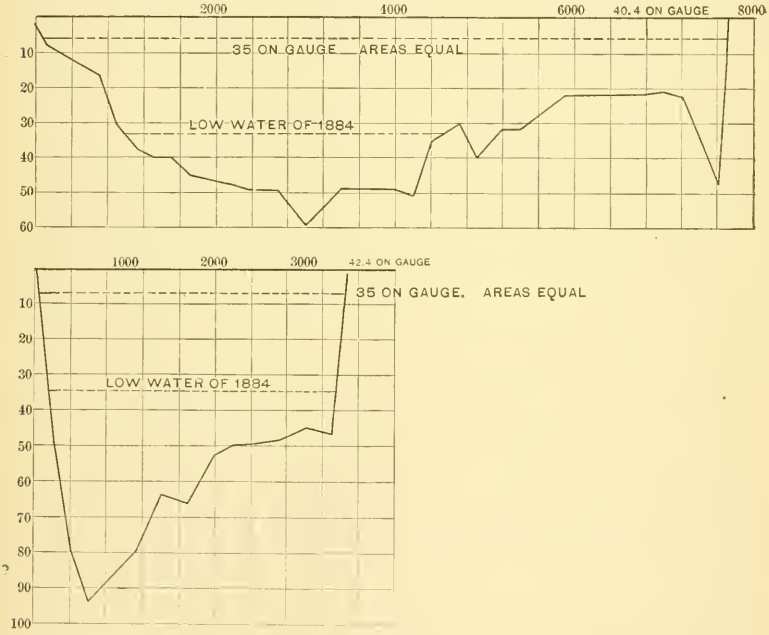


FIG. 21.

bar or in pool, becomes less than the one adjacent, its velocity relative to the latter is augmented, and scour takes place, accompanied by fill in the place where the current is slackened. This deduction is borne out by the recorded data. A gradual fill in the discharge section is seen at stages below about 34 or 35 ft., and this irrespective of rise or fall. There is every reason to suppose that the material for this fill is derived from scour of the bar above. At stages above 35 a tendency to scour is observed in the discharge section; not regular, however, but fitful. At the extreme high water of 1885, the areas of

the two sections, according to the survey of 1882, should be respectively about 227 600 and 201 300. The mean of these is about 214 500. To this limit, then, both areas would tend. Toward the climax of the rise the cross-sections are found to vary from about 207 900 to 218 600, the areas being maintained for several days above the mean, either from the passage of sand waves or other accidental circumstances, or more probably because the soundings of 1884–85, made at intervals of 50 ft., gave larger areas than those of 1882, at intervals of 250 or 300.

At a stage of 50 ft., the two sections should have areas of about 292 000 and 230 000; mean, 261 000. In 1893, the river actually reached this height. The datum area at the discharge section, which had been about 230 000 at a stage of 35, increased to 244 000 at 46, and then rapidly scoured out to 261 000 at 50, where it remained till the close of the observations.

Increase of Coefficient with Gauge Height.—In examining the curve of coefficients, they are seen to increase with the height of the stage, and this is generally understood to be the law of the progression. All formulas which do not make c a constant make it increase with either the velocity or the mean depth, or both. Is this true? And if so, why is it true?

First.—The roughness of the banks and bed cuts a somewhat greater figure at low water than at high. At a mean depth of 60 or 70 ft., it makes comparatively little difference whether the sides of the channel be rough or smooth. At 20 ft., its effect may be quite perceptible, though still small. According to Kutter's formula, at depths of 64 and 25 ft., there is hardly more than 1% difference in the coefficient of velocity for a considerable change in the coefficient of roughness.

Second.—At some stations, such as Paducah and Helena, there is an increased ratio between mean and bottom velocities at low water. If it be true, according to the supposition of M. Dupuit, previously quoted, that the velocity of the formula is the bottom and not the mean velocity, then the more nearly these quantities approximate, the more nearly will the formula give the mean velocity. Now the mean velocity is always the greater. To obtain it from the velocity of the formula a coefficient will be required, and this coefficient will be greater as the disparity between the mean and bottom velocities is more considerable. At Paducah the ratio between the maximum and

bottom velocities is, at 60 ft. depth, 0.47; at 10 and below, 0.76. At Helena, the same quantities are 0.58 and 0.71. The ratio between mean and bottom velocities is probably, for the mean of the two stations, about 0.60 at high water and 0.80 at low.

The observations at the other stations prove nothing. At Columbus there were only twelve measurements taken below 50 ft.; at Hays's Landing no bottom velocities were taken at all, and at Red River the record is extremely scanty and unsatisfactory.

Third.—The disparity between the general and local slopes is greatest at low water. For obvious reasons the discharges are always taken at narrow and deep sections. These become almost ponds at low water, with a great area and a low velocity, accompanied, of course, by a flat local slope. The computed or theoretical velocity, obtained from the general slope and the local mean depth, is, therefore, at such times excessive, and to reduce it to the observed velocity the coefficient must be made very small. On the other hand, at high water, the local slope approximates to the general, as has been explained. In deep sections the velocity at high water is often three or four times as great as at low, while in the wider places it is probably less than twice as much. On the bar above Arkansas City, in 1884–85, at high water, the mean velocity was probably about 4.8 ft., at low water about 3.5 ft. At the discharge section it ranged from 5.3 to 2.3. At Helena, in 1884, the velocity was 3.59 on the wide section when it was 2.85 on the narrow, at extreme low water.

On the other hand, at small depths in irregular sections, the coefficient may exhibit a considerable increase, for the reason formerly explained, namely, from the computation by the whole section instead of by partial sections.

Of these reasons the third has far more influence than all the others put together. It will therefore appear that the increase in value of the coefficient with gauge height is not a necessary incident, but is principally dependent on the shape of the section. Of this, abundant confirmation will be given.

Explanation of the Irregularities of the Curve of Coefficients.—It is now possible to explain all the vagaries of the curve of coefficients. The greater values on a rising river are due to the smallness of the areas, as in the case of Wilson's Point in 1890. The area cannot increase by scour until the area of the bar above exceeds it, and, hence, there

is a disproportionate augmentation of velocity above what would have been expected from theory.

The smaller values after the crest of the rise has passed are owing to the fact that the areas which have lately scoured out have not yet returned to their normal proportions, and hence the velocities are less than they should be. The areas cannot experience much fill until the cross-section over the bar has been materially reduced, a scour established there and a considerable slackening in the pool below to cause a deposit.

The remarkable perturbations that are seen in the upward branch of the curve at 36.5 to 39 ft. are due principally to the violent fluctuations in the area, to which reference has already been made, whereby a scour of 14 000 sq. ft. occurred in three days, followed by a fill of 8 000 sq. ft.

The velocity on January 13th was undoubtedly a good deal too small, as is evident from the fact that the area decreased at the same time, giving a discharge probably 60 000 less than it should have been. The velocity on January 16th is probably considerably in excess of the truth, as the area shows very little change, while the discharge is given as about 60 000 more than the mean of the preceding and following days, and greater than on any other day of the season. The notes say: "High waves."

In the lapse of time between January 23d and 30th, when no observations were taken, owing to the loss of the meter, a sudden and rapid scour began, which continued until February 6th, when it was succeeded by a fill. The record shows a sudden decrease in the velocity and in the coefficient, corresponding to the increase of area. A new meter came into use at this time, and, unfortunately, the observations before and after January 31st cannot be compared with any assurance of accuracy. A meter may be consistent with itself, but, unless with the best instruments and the most careful rating and manipulation, the results given by different meters must be expected to present some discrepancies. Judging from the record subsequent to February 12th, when still another instrument was substituted, the meter used from January 31st to February 12th gave velocities too low by 0.1 ft. per second or more.

Supposition of an Invariable Datum Area.—While the movements of the curve of coefficients have been accounted for, they have not been

quantitatively estimated, nor has any means been indicated whereby the coefficient at any stage, under any circumstances, may be ascertained.

In order to get rid of the perpetually recurring changes of area, the datum section at Arkansas City may be assumed to have been invariable for a certain period, say, during the rise in January, the water area being subject only to such changes as were produced by rise and fall. In such a case it is evident that the discharges would pass with certain velocities which might be very different from those which actually prevailed. Those velocities are easily ascertained, however, by computing the water area which would have prevailed if the section had been unaltered, and dividing the discharge by it. The area is easily computed by taking the difference between the proposed datum area and the actual datum area on any given date, and adding it to or subtracting it from the given water area. The standard area will be assumed at about that which prevailed during the greater part of the rise, namely, 212 000 sq. ft. The velocities which would have existed on that supposition are as follows:

Date.	Gauge.	Assumed area. Square feet.	Assumed velocity. Feet per second.	Assumed mean depth. Feet.	Theoretical velocity. Feet per second.	Coefficient.
December 31....	20.28	140 215	3.27	42.5	4.69	69.8
January 1....	23.33	150 335	3.78	45.5	4.92	76.9
2....	26.13	159 636	3.99	48.2	5.09	81.7
3....	28.71	168 222	4.28	50.6	5.74	78.4
7....	35.55	191 176	4.77	56.7	5.68	83.0
9....	36.85	195 530	4.96	58.0	5.76	86.1
10....	37.30	197 049	4.98	58.5	5.81	86.0
12....	38.10	199 751	5.09	59.3	5.81	87.6
13....	38.49	201 063	4.82*	59.6	5.85	82.5
14....	39.03	202 886	5.15	60.2	5.90	87.3
16....	40.28	207 106	5.39*	61.5	6.01	89.7
17....	40.61	208 219	5.18	61.8	6.02	86.1
19....	41.04	209 670	5.25	62.1	6.07	86.5
20....	41.20	210 210	5.10	62.5	6.07	84.0
22....	41.35	210 716	5.10	62.6	6.07	84.0
23....	41.48	211 155	5.18	62.7	6.08	85.3
31....	41.73	212 000	5.02	63.0	6.08	82.4

If these velocities are plotted to the gauge heights, a pretty regular line will result, considering the crudeness of the data, approximately straight, except at the top of the rise, when it begins to break a little (see Fig. 22). The velocities marked * are considered untrustworthy, on account of the large variations of discharge already alluded to. The observations from January 1st to 10th are not quite consistent

with one another, for they make a greater variation in the discharge at low stages than at high, the rise being still in active progress, which is never the case. The discharges for those dates should be about 531 000, 607 000, 684 000, according to the usual rate of progression of the discharge. If these figures are used, the assumed velocities for those dates will be as represented by a △ in the plot.

Slackening of Velocity at Crest of Rise.—In endeavoring to find a law for the upward progression of the velocity or of the coefficient, the data obtained near the top of the flood should not be used, as a

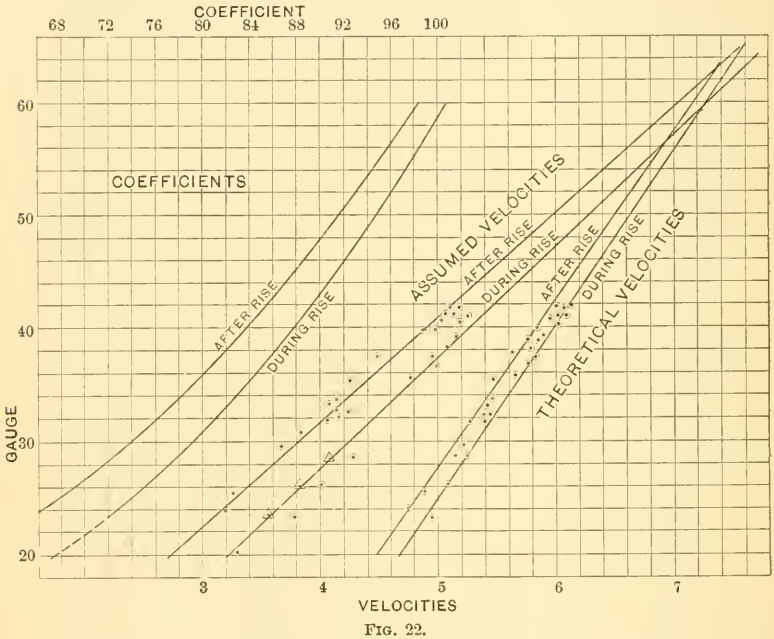


FIG. 22.

change in the condition of the river has already begun. It is a well-known fact, remarked long ago by Humphreys and Abbot, that a slackening generally occurs during the last foot or so of a rise, and that a less discharge is passed at the climax of the upward movement than at a stage a little lower. This fact is attributable to two causes.

First.—The slope begins to flatten at this time. As the rise falls off, the rate of progression of the gauges is diminished more rapidly above than below. It is rising at White River only 0.1 ft. while it is still rising 0.2 ft. at Arkansas City. Therefore the velocity also begins to decrease

even while the river is still rising. A cursory study of the slope curves on Figs. 16 and 17 will make this very clear.

Second.—There is a change occurring in the conformation of the river-bed. While the rise was in active progress, the bar was building and the pool was scouring. The scour did not begin in the pool, however, until the velocity in the latter surpassed that of the water on the bar, and the two processes continued until the two areas were equalized.

During this time, at all stages, the bars have acted as dams. If it had not been for them, the velocity in the pools, from the great depth prevailing in them, would have become excessive. The profile of the river has been somewhat as shown in Fig. 23. It is evident that the quantity of water which can be admitted to the pools is regulated by the height of the bars, and the conditions prevailing in the pools can only partially modify their effect. The velocity of the water at the wide sections is always greater than the theoretical velocity, because the local slope prevailing on the bar is in excess of the general slope. If the sections remained unaltered, a time would arrive when the general slope would be the greater. The rise produces a much greater proportional increase of mean depth on them than in the pools. At Arkansas City, for instance, a rise of 5 ft. above the gauge of 42.4 produces an increase of mean depth in the shallow section from 30 to 35, or one-sixth. In the deep section, the increase is from 60 to 65, or one-twelfth. Hence the velocity increases faster on the bars than in the pools, and at a stage of 55 ft. it would finally surpass the theoretical velocity. This process is modified by the alterations which take place in the bed. Not only is the pool scouring, but the bar is building; each pool furnishing the material for the accessions to the bar below it. While the upward progress of the river is tolerably regular and rapid, the daily increase of depth over the bar by virtue of the rise is more than sufficient to keep pace with the fill. When there is a considerable slackening in the rate of rise, however, as happens near the climax of every flood, the fill may exceed the increase in height of the water, and the bars may become veritable dams, increasing the depth in the pools while the discharge is the same, or maintain-

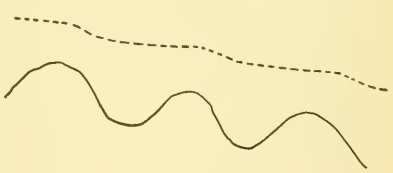


FIG. 23.

ing it when the supply of water is continually diminishing. For instance, from January 23d to January 31st, the cross-section at Arkansas City increased 5 500 sq. ft., or 1.8 ft. in mean depth. This is probably deposited in the first wide section below, and is sufficient to raise it a foot or so, while the gauge height has increased only a fraction of a foot.

Line of Assumed Velocities.—If this movement be general on all the bars throughout the river, as is likely to be the case, it may produce very little effect on the slope as estimated from station to station. Hence, in spite of the maintenance of the slope and of the discharge, there may be a slackening of the velocity; indeed there always is, and this is prolonged into the fall which succeeds.

A line drawn through the assumed velocities shown in Fig. 22 is very nearly a straight line; in fact a straight line fits them better than any other.

On the same supposition of an invariable datum area of 212 000, with the slopes actually prevailing, the theoretical velocities may now be computed and plotted, that is, the products $100\sqrt{rs}$, and the coefficients required to bring them to the assumed velocities lately found. If the slope remained constant at all stages, the line of theoretical velocities would be a curve concave to the axis of ordinates. The slope, however, at this station increases with the gauge height; consequently, the theoretical velocities are found ranged, as nearly as may be, in a straight line, not parallel to the line of assumed velocities, but inclined to it and intersecting it as the gauge gets higher. By means of these two lines, the coefficients of any number of points may be determined, and they will be found to correspond pretty closely with those given on page 445. They form a curve, given also in Fig. 22. The lines can be extended to any stage, and the coefficient ascertained. By taking the slope from the slope curve, and the mean depth from the known addition to the gauge height, the velocity may be computed. The assumed cross-section may also be easily ascertained by adding the increase of area due to the given rise, to the datum area of 212 000. The discharge at the gauge height required may hence be easily ascertained. The velocity may be more easily found by simply prolonging the straight line representing the assumed velocities.

Care should be taken, however, in extending by analogy the conditions found at moderate stages to extreme high water. Usually this

is the best means known for arriving at a correct estimate of the conditions likely to prevail at such stages, the direct observations obtained at flood height being generally tainted with many elements of error, which it is very difficult to eliminate. All the checks that are available should be used, however.

Discharge Curves.

The ordinary method of arriving at the discharge at stages beyond those directly experienced is by prolonging the curve of discharges. Now a very superficial examination of any discharge curve will show a few leading points. The discharge, as a general rule, increases with the gauge height, and in an increasing ratio, which, if the bed of the river remained unchanged, would probably be tolerably regular. The rate would always be greater on a rise and less on a fall, from the known peculiarities of the slope. For this reason it has been considered proper, as has been already observed, in tracing a discharge curve, to select only times when the river was stationary, and thus eliminate this disturbing influence. It is found that this process gives a far more regular curve, yet there are still very great discrepancies observable; so persistent, and recurring with such regularity, that it is impossible they should be errors of observation. Even in the measurements of any single season, they are conspicuous, if the time of observation be at all prolonged; but they are most noticeable in comparing the data obtained from different years.

It was observed several years ago by Colonel Suter,* that in every series of discharge observations, no matter how scattered they may appear, it was possible to group them about a series of curves parallel to a certain standard curve. That is, the curve of one year or of one stage would not answer for another year or another stage unless its position were shifted up or down. This fact is strikingly visible in every series of observations, from the days of Humphreys and Abbot to the present time. The word "parallel" must not be understood always in a literal sense, for each curve is more or less affected by perturbations of one kind or another. The general effect is the same, as Colonel Suter remarks, as if the whole river were bodily raised or depressed.

* Paper on investigation of discharge measurements, in "Report of the Mississippi River Commission for 1891," p. 3419.

Discharge Curve of 1884-85 at Arkansas City.—Fig. 24 shows the principal features of the discharge curve of 1884–85 at Arkansas City. It will be seen that the curve consists of two principal branches, one representing the main rise of the season, in January, 1885, the other the fall which succeeded that rise and the movements subsequent thereto. It is evident that during the latter period almost the same relation prevailed between the successive values of the discharge as during the rise, the difference being that the discharges passed at

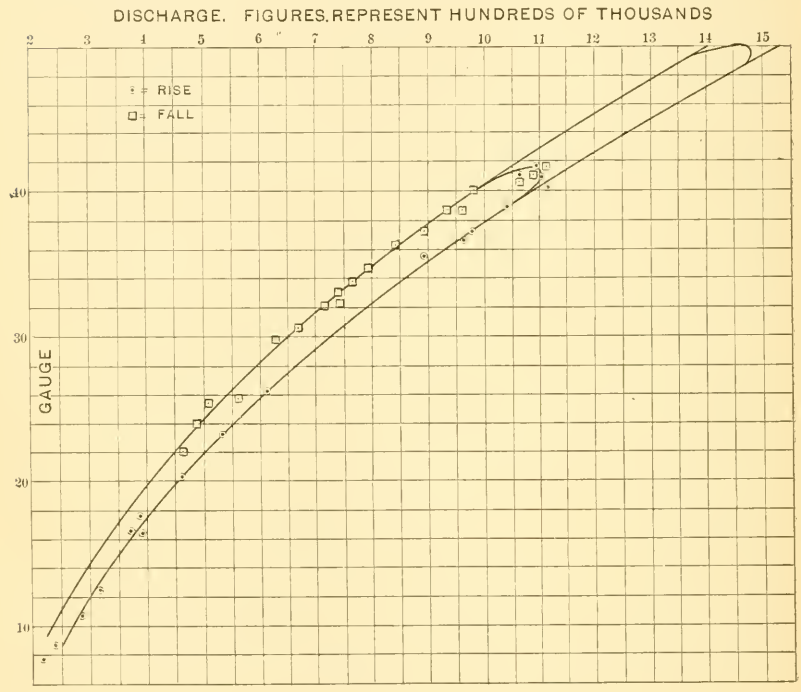


Fig. 24.

a gauge reading higher by a little more than 2 ft. after the crest of the rise had passed than during the progress of the swell. The two curves are very nearly parallel.

Now the discharge at any height is equal to the assumed velocity multiplied by the water area corresponding to that height, as computed from the standard datum area, supposed to be invariable.

At medium and high stages the area increases with great regularity. At 20.28 ft. the width at the water line was 3 299 ft. At 41.7

it was 3 380. The increase of area, therefore, for each 2 ft. of rise would be successively 6 608, 6 616, 6 624 ft., etc., so small a difference that if the values of the areas were plotted to the height as an argument to any ordinary scale, the line drawn through the points thus established would not be distinguishable from a straight line. But if the assumed velocities also range themselves in a straight line, the values of the products of the two should be represented by a series, the differences of which should run only to the second order. This in fact is the case. If the ascending and descending values of the discharge be arranged in two series, they will run very nearly as follows:

	During Rise.			After Rise.		
Gauge.	Discharge	First difference.	Second difference.	Discharge.	First difference.	Second difference.
24.....	550 000	.....	.....	485 000	.....	.....
26.....	604 000	54 000	.....	535 000	50 000	.....
28.....	661 500	57 500	3 500	588 500	53 500	3 500
30.....	722 500	61 000	.....	645 000	57 000	.....
32.....	787 000	64 500	.....	705 500	60 500	.....
34.....	855 000	68 000	.....	769 000	64 000	.....
36.....	926 500	71 500	.....	836 500	67 500	.....
38.....	1 001 500	75 000	.....	907 000	71 000	.....
40.....	1 080 000	78 500	.....	981 500	74 500	.....
42.....	1 162 000	82 000	.....	1 059 000	78 000	.....
44.....	1 247 500	85 500	.....	1 140 500	81 500	.....
46.....	1 336 500	89 000	.....	1 225 000	85 000	.....
48.....	1 429 000	92 500	.....	1 313 500	88 500	.....
50.....	1 525 000	96 000	.....	1 405 000	92 000	.....

In this table the discharges are extended to a stage 10 ft. beyond that at which the observations terminated. If it is assumed that the behavior of the several quantities considered would be the same at higher stages as at the stage actually reached, then the velocities would follow the line pursued during the rise, until a short time previous to the culmination of the flood, when the velocities would break away, remain stationary for a while, the rise still progressing slowly, and then gradually slacken off; and during the fall they would approximately pursue a prolongation of the line which they followed during the same circumstances at the lower stage. If a line were drawn parallel to the two curves which have just been determined and passing through the point representing the discharge at the highest gauge reading attained, namely, 41.7, then the intersection of this curve with the horizontal line representing the height of 50 ft. would give the discharge for that reading. The mean of six observations

taken at the top of the flood gives a discharge of about 1 086 000. From either of the two series given on page 451 the increase of discharge corresponding to that value may be ascertained for an addition to the height of 8.3 ft. From the first series it is found by interpolation that a discharge of 1 086 000 corresponds to a height of 40.15. A height of 48.45 demands a discharge of 1 450 000. Then the increase of discharge is 365 000, and the discharge at the height of 50 ft., supposed to be at the stationary point, or at the turn of the curve, will be 1 450 000.

Changes of Plane.—The phenomenon of the shifting of the stage of the discharge is very appropriately called by Colonel Suter a "change of plane." It seems to be principally due to the changes which have been noted as occurring in the river-bed from stage to stage. The fill on the bars continues steadily, in harmony with the upward progress of the river, until, by the joint effort of the fill and of the contemporaneous scour in the pools, the areas, and hence the velocities, are brought to a state of equality. So during the progress of a fall, after the medium stage, or stage of equality of areas, has been passed (which at Arkansas City in 1884–85 seems to have been about 35 ft., varying, of course, with the changes of the bed), the wide places cut out and the pools fill, with a tendency toward the same result, equalization of cross-section and of velocity.

At high water, owing to the energy of the forces at work, the process goes on with great rapidity. At low water, the velocities are feeble and the changes from day to day comparatively small. If the season be short, the equilibrium of areas may never be reached. Probably it seldom is.

The medium stage would appear to be, on the whole, one of conservatism of existing conditions, whatever they may be. If this stage succeeds a considerable and sustained rise, it finds the bars high and the pools deep. Coming after a prolonged low stage, it finds the bars low and the cross-section in the pools small.

It has been shown that the bars are the controlling elements of the bed at all stages. A general elevation of the bars throughout the river, or throughout that portion in which the elements are considered, is equivalent to a general elevation of the water surface. For any one year, the oscillations of the datum area at the discharge section are generally pretty fair exponents of the changes from stage to stage on

the bars, the movements of the latter being in the inverse direction. For instance, in 1884–85, the continued decrease of area during the low stage indicated a progressive scour on the bars, and hence a "change of plane" in a downward direction; while at the top of the rise the rapid enlargement of the discharge section was significant of a fill in the wide places, and hence of a rise of the water plane.

From year to year, the phenomena have not necessarily the same significance. Thus, in 1889, with a datum cross-section of only 195 000, the discharge of 745 000 passed at a stage of 35 ft. In 1885, after the rise, with a datum area of about 212 000, the same discharge had passed at a stage of about 33, and during the rise, with an area of 206 000, at about 31. Hence it is probable that there are other changes of the bed, such as have been mentioned on pages 428 and 429, of a more permanent or perhaps of a periodic nature, which have not yet been carefully studied. Such changes may be supposed to proceed from a long continuance of high-water or low-water conditions; from an unusual predominance of floods from clear-water streams like the Ohio or upper Mississippi, or from silt-bearing tributaries like the Missouri and Arkansas, all which may very readily have an important effect upon the general bed, etc. Some importance may also be attributed to the maintenance of levees and the consequent confinement of floods, or, on the other hand, to the occurrence of crevasses and the deterioration of the bed which may naturally follow therefrom. The reclamation of tracts hitherto unprotected and the consequent exclusion of the action of basins as reservoirs may be included in the same category. These are very complicated matters, and require for their discussion much local knowledge and much careful study.

The respective parts played in the "change of plane" of the discharge by the several causes may be easily discriminated. In the case of 1884–85, the slopes corresponding to the two branches of the curve differ by an extreme quantity of about 0.000004. At a stage of 35 ft., this difference would produce a change in the velocity of about 0.1 ft. per second, or in the discharge of about 20 000 cu. ft. The actual difference was about 90 000. The change of area in the discharge section alone would not affect the discharge, but only the velocity. Hence the elevation of the bars was responsible for more than three-quarters of the discrepancy.

The change of plane is sometimes mostly local. On March 9th to

11th, 1893, a discharge estimated in the mean at about 1 050 000 passed Arkansas City at a stage of 43.1, while in 1885 the same discharge passed between 39 and 40. In this instance the change of plane was confined principally to Arkansas City, the gauge at that point being much higher, relatively, to White River and Greenville than in 1885. At the height of the rise in the latter year, the following figures show the readings of the gauge at White River, Arkansas City, Greenville, Lake Providence and Vicksburg, compared with March 9th to 11th, 1893.

	1885.	1893.
White River	43.60	44.3
Arkansas City	41.70	43.2
Greenville	37.60	37.7
Lake Providence	34.75	35.8
Vicksburg	42.40	42.4

Thus, it will appear that there were slight changes of plane at White River and Lake Providence, a considerable one at Arkansas City, and none at all at Greenville or Vicksburg.

High-Water Conditions Should Repeat Themselves.—If the high water be of long duration, the slope should be brought to very nearly the same value from year to year, the depths of water on the bars should be nearly the same, and, in short, the conditions should repeat themselves, except in so far as they are affected by permanent changes of the bed. Suppose the bed of the river to be as represented by the line *C D E F* in Fig. 25, and suppose the water surface

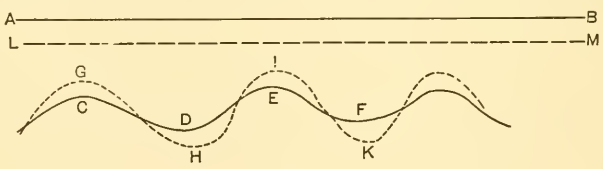


FIG. 25.

at the height of the flood to be as represented by *A B*. The area on the bars being much greater than in the pools, the velocity will be slackened and the bars will build. In the pools the velocity will be increased and the bottom will be depressed. The bed will then assume the shape *G H I K*. Now this reasoning is altogether independent of the exact shape of the original bed *C D E F*. It may be as it is figured in the sketch, or it may be already in a position approximating

that shown by *G H I K*. In the latter case there will be little change. If the river remain at flood height for a long time the result will be the same in the one case as in the other.

The end of the process will be reached when the areas at *G* and at *H* are equalized. But suppose the water level to occupy the position *L M*, lower than the normal, instead of *A B*; will not, then, the areas be equalized and the movement of material stop prematurely at a lower level than before? It appears that this should not occur unless the whole bed of the river undergoes a permanent change. The high-water slope is fixed or oscillates between limits. As long as the general bed of the river remains the same, and the movement of material is confined to the transference of silt from one locality to another nearly adjacent, the high-water slope will not be disturbed thereby, provided it has time to adjust itself. As the bars rise the water will rise also until it has attained its normal slope. If it be supposed possible that the process should extend further than this and the bars rise still more, the consequence would be that the level above the bar would be raised and the velocity over the bar increased, thus putting an end to the building process. In other words, as it has been expressed, "the slope would prove more inflexible than the bottom."*

The higher the flood, the more rapidly and certainly will the normal level be reached. If the flood be short and of moderate height, it is evident that the water plane at the extreme point reached may be above or below the normal, the conditions at medium stages having comparatively little tendency to produce change. The stage of water will then be largely controlled by the condition of the bed as the rise finds it. With the powerful forces prevailing at extreme flood, adjustment is speedily brought about. If a high water indefinitely prolonged is assumed, it appears a natural consequence that a certain height, slope and other conditions should be reached which would be reproduced under similar circumstances at other times, with the greater certainty and precision as the new conditions conformed to the old, and as the time was extended for their operation.

It would be desirable to bring this reasoning to the test of fact, but this cannot be rigorously done, for the true high-water slope of the Mississippi has never been ascertained, even for one year. It has always been disturbed by the same influences that have introduced so

* "Report of the Mississippi River Commission for 1884," p. 2551.

many complications into the movements of the discharge; escapes and returns of water, influence of tributaries, reclamation of lands, etc. It is also very much affected by duration of flood. Every one has remarked, however, the persistence with which high-water conditions repeat themselves. Any "river prophet," if given the stage at Cairo and its duration and the condition of the tributaries, will state within 0.5 ft. or less what height will be reached at Helena, Arkansas City, Greenville or Vicksburg from a comparison of previous experiences.

Extreme Discharges at Arkansas City.—The discharge at an extreme height has been measured at Arkansas City under very favorable conditions. The stage was 49.6, within 0.7 ft. of the highest ever reached there. The river was stationary. The mean of the observations of eight days are available, which are wonderfully accordant. There was no disturbing influence within 60 miles below, and none at all above. The measurements had been checked, up to a few days before, by simultaneous observations at Wilson's Point, 93 miles below, and by the free use of the flanking method at both places. The discharge was also observed at the extreme high stage, 50.3, which was reached the same season; but at that time there were disturbances which rendered the determinations of the discharge less trustworthy, and which palpably affected the gauge height.

The mean discharge for eight days, at a reading of 49.6 on the Arkansas City gauge, was 1 425 206. As computed from the series on page 451, from the data of 1884–85, the discharge at this stage should be 1 431 478.

In comparing the two series just referred to, the first differences represent the increase of the discharge for each 2 ft. of rise. While the two curves are parallel the same differences do not correspond to the same gauge heights, but to the same values of discharge. Thus, in the ascending series, the increase of discharge from 30 to 32 ft. was 64 500. In the descending scale (or rather after the fall) the difference 64 500 does not correspond to the stage of 30–32, but about to that of 32–34. It corresponds, however, in both to a discharge of about 720 000. Hence, in using this method, discharge, not gauge, is to be considered.

The discharges observed at 49.6 may be assumed to be nearly correct. The discharge for any higher stage not too remote may then

be obtained with a very fair approach to accuracy, by applying the analogies deduced from the series on page 451, especially as they have given pretty nearly the correct result above, though extended over nearly 8 ft. of rise. At 50 ft., there should be about 1 445 000; at 50.3, 1 458 500. The mean for six days of the measured discharge actually taken at the latter stage, was about 1 504 000. This includes the extraordinary measurement of 1 667 000. Excluding this, the result was about 1 480 000. Probably this is not far from the truth, but the stage would have been a little higher had it not been for disturbances.

At the stage of 53.3, which is the grade of the present levees, the discharge would be (still starting from 49.6) 1 605 600; at 56, about 1 745 000.

It is now allowable to return to the method of assuming an invariable area and deducing velocities therefrom, which there was doubt about extending to high stages without some check. If the line of assumed velocities is developed beyond the time of the culmination of the rise, and it is followed during the fall, it will be found that this line, like that of the discharges, is approximately parallel to the ascending line (see Fig. 22). It is hardly worth while to give the table from which the several points of the line were derived. If now a straight line is drawn parallel to these lines through the highest point reached in 1885, namely, 41.7, the assumed velocity at any stage will be obtained. At 49.6 it will be about 6.00. The width will be taken at about 3 400. At 41.7 it was 3 380. The increase of height is 7.9 ft.; increase of area, 26 860. Adding this to the assumed datum, 212 000, the assumed area will be at 49.6, 238 860, and the discharge, 1 433 160. At 50.3, the assumed area will be 241 240; velocity, 6.07; discharge, 1 464 327. At 53.3, the area will be 251 440; the velocity, 6.40; the discharge, 1 609 216; at 56, the discharge will be 1 746 154.

Observations of 1893.—The discharge observations of 1893 extended over only a small period, namely, from March 9th to April 8th, and from May 5th to June 7th, covering a range of only 16 ft. on the gauge, from 34.2 to 50.3. For general study they are therefore not so instructive as the series of 1884-85. They have, however, the great advantage of being known to be, in the main, tolerably accurate. They were accompanied by simultaneous observations at Wilson's Point, which serve as a valuable check, especially before crevasses

intervened. They have another advantage, that they were taken at very high stages. It has always been a matter of doubt how far inductions made from medium stages were applicable to extreme high-water conditions. These observations, too, exhibit considerable variations in the cross-section. The datum is taken at 50.3 instead of at 41.7. The datum areas range from 233 252 to 262 008. The standard area will be taken at 242 000, which corresponds pretty nearly to the datum of 212 000 in 1884-85. The data observed during the rise in May are too few, however, and extend over too small a range of the gauge to be a safe guide. The fact that the observations were not continuous prevents tracing the various changes of the bed, although they can be easily imagined. In the considerable discrepancies of the recorded velocities and discharges, it will be safest to take the means of those which were observed while the river was stationary for a considerable period. There were two such times, corresponding to stages of 40.0 and 49.6 respectively. At these dates the following data are found:

Date.	Gauge.	Assumed area.	Assumed mean depth.	Assumed velocity.	Discharge.
March 20th-29th..	40.0	207 479	60.5	4.55	943 931
May 15th-23d.....	49.6	240 022	70.6	5.94	1 425 206

If the line of assumed velocities is supposed to be straight, as before, the velocity at 50.3 will be 6.05, and the discharge 1 464 100. At 53.3, the velocity will be 6.48 and the discharge 1 635 325. The observations actually taken at this stage have already been given. A comparison of the Wilson's Point observations shows no such fluctuations of the discharge as are recorded at Arkansas City. There were crevasses at this time between the two stations, but they had been running for two weeks or more, and there was little change from day to day in their rate of discharge. There should have been then nearly a constant difference between them equal to the combined discharge of the crevasses. The latter were measured twice during the season, but the results are not entitled to much confidence, the conditions being extremely unfavorable and the methods somewhat crude. A crevasse is divided into several sections; at least three direct flows and two reverse flows, caused by eddies, as shown in Fig. 26; and it is extremely difficult to measure their discharge correctly. The discharge at Wil-

son's Point at this time, for the five days succeeding the excessive velocities recorded at Arkansas City, averaged 1 391 658, and for the five days preceding, 1 412 214.

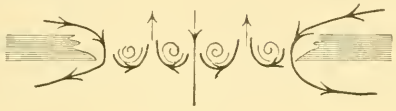


FIG. 26.

Use of Method of Coefficients.—The use to be made of the method of coefficients is to be considered next. In 1884–85, the progression of the theoretical velocities before and after the rise followed nearly straight and parallel lines. So the assumed velocities, on the basis of a constant datum area, followed lines which were nearly straight and parallel, but inclined to the lines of theoretical velocities. The coefficients, when plotted, form curves which have the same general form, but are not exactly parallel (see Fig. 22). In each curve, however, the progression of the coefficients follows very nearly the same law, as might have been expected. The following table gives the values of the coefficients arranged in series, with the differences.

	DURING RISE.		AFTER RISE.	
Gauge.	Coefficient.	First difference.	Coefficient.	First difference.
24	72.4	.....	65.8	.....
26	74.8	2.4	68.6	2.8
28	77.0	2.2	71.1	2.5
30	79.0	2.0	73.5	2.4
32	81.0	2.0	75.8	2.3
34	82.8	1.8	77.8	2.0
36	84.6	1.8	79.8	2.0
38	86.3	1.7	81.8	2.0
40	87.8	1.5	83.7	1.9
42	89.3	1.5	85.5	1.8
44	90.8	1.5	87.1	1.6
46	92.2	1.4	88.7	1.6
48	93.6	1 4	90.2	1.5
50	94 9	1.3	91.7	1.5
52	96.2	1.3	93.2	1.5
54	97.4	1.2	94.5	1.3
56	98.5	1.1	95.8	1.3
58	99.6	1.1	97.1	1.3
60	100.7	1.1	98.4	1.3

At 49.6 ft., in 1893, the coefficient deduced from actual observation, with the assumed data, given on page 458, is 87.6. From either series of coefficients given above, it is seen that a coefficient of this value increases, for an increased height of 3.7 ft. (that is, from 49.6 to 53.3),

by about 2.7. That is, at 53.3 it would become 90.3. The slope would have been about 0.0000651. The mean depth, to datum of 242 000, would have been 74.3. Hence, the assumed velocity would have been 6.28; the area, 252 710; the discharge, 1 587 019.

If, instead of taking an assumed datum area, the observed area, namely 253 774 (mean of 9 days), had been taken, the observed mean depth, 73.3; the observed slope, 0.0000655, and the observed velocity, 5.625, a coefficient of 81.2 would have been obtained, which, at 53.3, should have increased to about 84.5. This should have given a velocity of 6.00, an area of 266 558, and a discharge of 1 599 348. The observed slope was a little greater than the normal for the stage, on account of crevasses below Greenville, which lowered the water line at that place more than at Arkansas City.

As Humphreys and Abbot would say, no closer agreement could be desired, and it is possible that the use of local coefficients may be found serviceable for purposes of approximation or even of accuracy. It would be very rash, however, to come to such a conclusion without further and much more extensive trial. It will appear presently that there must be a separate series of coefficients for each place, which will be subject to variation by any change in the location of the cross-section or any considerable modification of its type. Nowhere are hasty generalizations more unwise, and nowhere are they more frequently brought to grief, than on the Mississippi.

Increase of Coefficient Due to Shape of Section.—It has been remarked that one cause of the increase of the coefficient with increasing gauge height is the deep and narrow conformation of the cross-section. So influential is this cause that all others are as nothing in comparison with it. If, instead of taking the discharge section at Arkansas City, the wide section or bar above had been chosen, a series of coefficients would have been had that moved in an inverse sense, and decreased as the river rose. As this is an important and interesting phenomenon, a few examples may be given. The dimensions of the two sections are taken from the surveys of 1882, and the plottings made therefrom are shown in Fig. 21. As there are no means of ascertaining the changes that occurred from day to day, it will be assumed that the bed of the river underwent no change during the alternation of high water and low water. Whatever change took place would affect the figures only, not the reasoning. The slopes were taken from the mean

results of the observations of 1884-85; the discharges from the mean discharge curve of several years. For the wide and narrow sections respectively the following results are found:

Stage. Feet.	Areas. Square feet.	Mean depths. Feet.	Slope.	Theoretical velocities. Feet.	Discharge Cubic feet.	Actual velocities.	Coefficients.
7.4. Low water of 1884....	51 600 and 86 500	16.8* and 27.0	0.0000375	2.51 and 3.18	221 000	4.29 and 2.56	171 and 80.5
20.....	111 600 and 128 500	21.3 and 39.2	0.000049	3.23 and 4.38	440 000	3.94 and 3.43	122 and 78.4
30.....	139 700 and 161 400	19.1† and 48.0	0.000054	3.21 and 5.09	650 000	4.66 and 4.03	145 and 78.4
35	178 400 and 178 400	23.2 and 52.7	0.000057	3.63 and 5.48	820 000	4.66 and 4.66	127 and 85
42.4‡.....	233 000 and 203 700	30 and 59	0.000059	4.21 and 5.90	1 110 000	4.76 and 5.46	113 and 92.6
50.....	292 000 and 230 000	37.6 and 66.6	0.0000625	4.85 and 6.45	1 450 000	4.97 and 6.30	102.5 and 97.6
55.....	330 750 and 247 250	42.6 and 71.6	0.000064	5.20 and 6.74	1 687 000	5.11 and 6.82	98.2 and 101.2

* In taking the mean depths at low water in the wide section, only the principal part of the section was considered, the isolated pool being left out of account.

† The mean depth is here actually less than a stage 10 ft. lower, because in the meantime the river has spread over a bar 1 500 ft. wide.

‡ The surveys were made at this stage, which corresponds nearly to "bank-full."

Section of Uniform Coefficient.—From these examples it is evident that the commonly accepted idea, on which so many formulas have been based, that the coefficient always increases with the height of the river, or with mean depth, is not a law at all, but in part, at least, a peculiarity attached to narrow sections. A section may be imagined, a sort of mean between these two types, in which the coefficient should remain uniform at all stages from the highest to the lowest. It is indeed possible to construct such a section; it has been done, approximately, in Fig. 27. It is evident that such a section may be conceived as existing at any point in the reach under consideration, and that if it so existed, the discharges would pass at the velocities corresponding to those given by the formula $v = 100\ \sqrt{rs}$, and at the stages indicated

on the margin. This imaginary section might be used in estimating discharges for higher stages than have been observed. It is a reasonable supposition that at 53.3 ft., for example, the same relation would exist that has prevailed from the stage of 30 to that of 50; that the mean depth of the imaginary section would be 54.5, the slope about 0.0000635, that the velocity would hence be 5.86, the area 272 600 and the discharge about 1 603 000. It is very probable, indeed, that these figures would nearly represent the actual area, velocity and discharge at the Arkansas City section. The neutral section, or section of con-

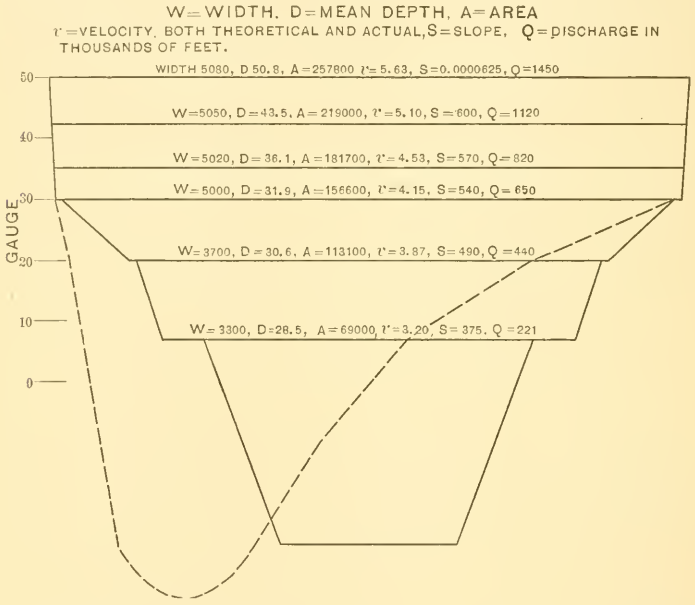


Fig. 27.

stant coefficient, is very nearly a mean of the wide and narrow sections, and is therefore such as would be reached at the top of a great rise, when it had been prolonged sufficiently to bring about an equalization of areas. Indeed, the neutral section is not far from being about the mean section of the river.

For the present purpose, it is hardly worth while to construct such a section with the last degree of accuracy. The shape of the section will depend on the values given the discharges at different stages, and if such values are regarded as satisfactorily determined, higher values may be obtained by arranging the known values in a series and deter-

mining the law of their progression, as has already been done, without resort to an imaginary section.

Discharges at Wilson's Point.—Having examined the data available for the purpose at Arkansas City, Wilson's Point may be taken up. Here there is the disadvantage of having no continuous series of observations extending over low as well as high stages. If an attempt is made to supply the missing data at the lower end of the discharge curve by transferring the Arkansas City discharges of 1884–85, taken at stationary stages, to the gauge readings at Wilson's Point (or rather Lake Providence) at those times, there will be no longer the check which would have been afforded by a series of independent observations, and the results will be substantially identical with those just

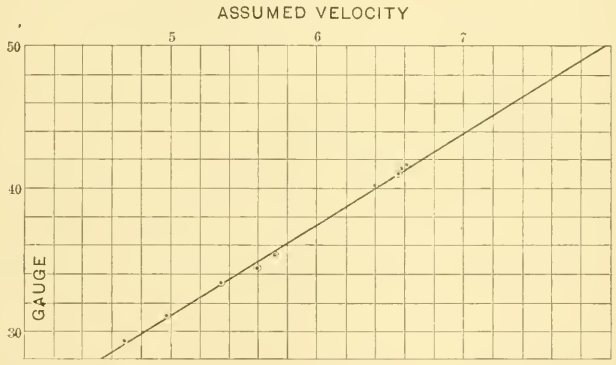


FIG. 28.

obtained. In 1882, there were discharges taken at Hays's Landing, or rather Bass's Landing, 10.4 miles below Lake Providence, which are easily transferred to that point, and are thus comparable with those taken at Wilson's Point, which are referred to the same gauge. The Hays's Landing observations, however, are of little avail, because all the discharges taken at high water were vitiated by the disturbances arising from crevasses and return flows in a wonderful manner. Only about half of the discharge of the river was retained in the channel at that point. The only records which hold out any promise of usefulness are those of 1893, and they have a very limited range.

A datum area of 210 000 may be assumed at 41.8 on the Lake Providence gauge, and the velocities thus obtained plotted to the gauges. The results are not at all accordant, and exhibit discrepancies where

there is no apparent reason for them. For instance, when the river is stationary for several days, and there is no material change of slope, the recorded discharge fluctuates between 972 000 and 930 000, and the reduced velocity between 5.48 and 5.23. Under these circumstances it was thought best to divide the observations into groups and take the means. The result of this proceeding is shown in Fig. 28. The velocities are best represented by a straight line, which, being prolonged, cuts the line of 42.8 (which is supposed to be about equivalent to 50.3 at Arkansas City) at 6.82. This velocity, with the area of 216 580, gives a discharge of about 1 459 000. At 45.7, which may be supposed to be about equivalent to 53.3 at Arkansas City, the velocity will be 7.3; the area, 225 100; and the discharge, about 1 643 000. The observations, however, are too few and their range too small to reason from them with confidence for any considerable interval. It is not certain that 45.7 at Lake Providence would be the equivalent of 53.3 at Arkansas City, for the gauge relations of the two differ widely at different stages, and are not at all consistent.

On the whole, the discharge which passed Arkansas City and Wilson's Point in 1884-85 at a stage of 41.7 on the gauge at the former place and 34.7 at the latter, was about 1 086 000 ft. per second; in 1893, at 50.3 and 42.8* respectively, about 1 460 000. At a stage of 53.3 and 45.7, or whatever is its equivalent at Lake Providence, there would have passed something more than 1 600 000.

These estimates are conservative, and it is probable that the changed conditions which prevail at stages over bank-full would operate so as to increase the discharge by a noticeable amount, perhaps as much as 50 000 cu. ft., as will be shown hereafter.

Maximum Discharge.

It now remains to endeavor to determine the maximum discharge of the river, and the heights which it would have attained at the principal stations had it been confined. For this purpose there are available the observations of 1882, taken at Paducah, Columbus, Helena, Hays's Landing and Red River Landing, and the high-water discharges of the years 1890, 1891, 1892 and 1893. Measurements were made in 1882, 1883 and 1884 of the discharge over banks and through crevasses, and crevasse measurements have been made during each of

* The highest point actually reached was 41.8. Allowance is here made for the effect of crevasses, though the determination is not very certain.

the years of great flood. Of the observations of 1882, the only ones which are available for the purpose of determining the maximum discharge are those of Columbus and Helena.

At these stations, the velocities at extreme high water, in 1882, were usually determined by rod floats.

At Columbus.—At Columbus the length of the rod at high water was generally 37 ft. The mean depth at this stage was about 70 ft., and the maximum about 100 ft. The Francis formula was applied in correcting the measurements, and this would give a coefficient of reduction of about 0.93, for the observed velocities. From the shape of the Columbus

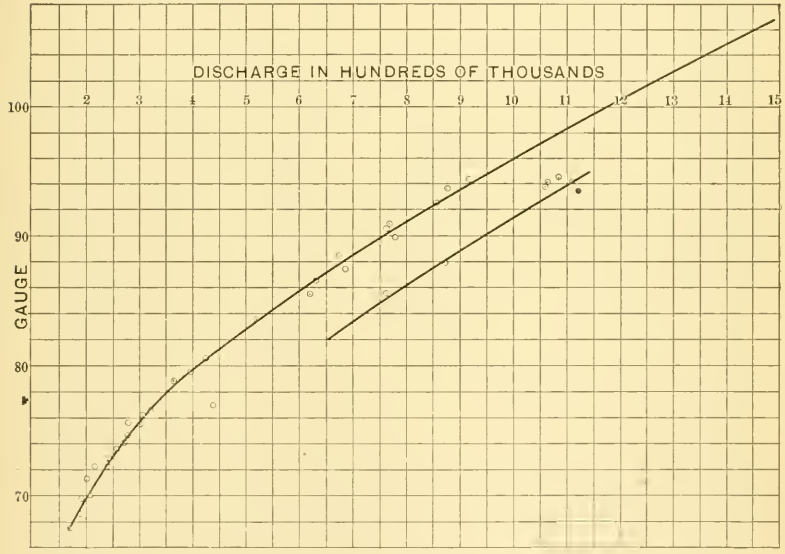


FIG. 29.

curves of vertical velocity at high water, the proper ratio would be about 0.91. Now the highest observed discharge was 1 603 215, on a rising river, at a stage 1.5 less than the maximum. A few days later, after a fall had set in, a discharge of 1 558 323 was recorded, at a stage only 0.17 less than the maximum. The slopes in these cases were 0.0000895, 0.0000960 and 0.0000913, so there seems to be no reason why the first discharge should have been in excess of the last.

If the discharge curve is plotted, it will be found that the data for the ascending branch are very few (see Fig. 29). So far as they go,

they correspond very fairly to those of the descending branch, which are pretty numerous, and afford the means of determining the curve with reasonable accuracy. Here, as at Helena, later, the means of the observations near the same stage, on a falling river, have been taken. It is a little flatter than the Arkansas City curve. The low-water part of the curve seems to follow a different law of progression from that of the high-water part, a frequent occurrence; and above the overflow line, which is fixed by the observers at 95 ft. on the gauge, there is still a different law, from causes which are now clear.

The series of values found for the branch of the curve pursued after the rise are as follows:

Gauge.	Discharge.	First Difference.	Second Difference.
78	355 000	.....	.....
80	415 000	60 000	.....
82	478 000	63 000	3 000
84	544 000	66 000	.....
86	613 000	69 000	.....
88	685 000	72 000	.....
90	760 000	75 000	.....
92	838 000	78 000	.....
94	919 000	81 000	.....
96	1 103 000	84 000	.....
98	1 090 000	87 000	.....
100	1 180 000	90 000	.....
102	1 273 000	95 000	.....
104	1 369 000	96 000	.....
106	1 168 000	99 000	.....

Above 95, a well determined series of measurements gives the increase of discharge from the stage of 100 to that of 102 as about 130 000 cu. ft. Taking the mean of nine observations at the nearly stationary stage reached from January 25th to February 6th, it will be found that a gauge reading of 99.96 corresponded to a discharge of about 1 394 000. At this rate, the discharge at the highest point reached, 102.8, should be about 1 573 000. This value depends, of course, on the correctness of the determinations. If the errors in them are merely in the nature of fluctuations, probably they will be corrected by using means, as has been done. If the quantities are all too large or too small, owing to common sources of error, the result will be affected in the same way. Relatively to one another, the observations seem to be fairly consistent.

It was estimated that a considerable volume of water escaped over the banks, and did not pass Columbus in the channel at all. The

measurements of this quantity made in 1882 and in 1883 are so discrepant, and the matter is so surrounded by complications, that it can be discussed more conveniently in connection with the subject of escapes and returns from reservoirs or across points.

It is now in order to see how these conclusions tally with others, made at different times and with more accurate means. In the floods of 1892 and 1893, high-water discharges were taken at Columbus, and in the latter year at New Madrid also. In 1892, the Cairo gauge reached 48.3, and in 1893, 49.3, respectively 3.6 and 2.6 ft. less than in 1882. In 1892, the mean for the five highest days was 1 379 770. The gauge reading for the same time was 43.0 on the Belmont gauge, equal to 101.0 on the old Columbus gauge. In 1893, the mean of nine days was 1 493 507, for a gauge height of 43.7 at Belmont, and of 49.2 at Cairo. At New Madrid, the discharge for a mean of seven days, was 1 474 048. The observations at Columbus were taken with double floats at mid-depth, and are, no doubt, too large. Even at New Madrid the same remark may hold good. The estimate of the discharge given above is the mean of the meter and the float observations. The meter was placed at 0.6 depth and the floats (double) at mid-depth, and no correction was applied for either. The velocities found by floats, however, were almost uniformly the smaller of the two.

For a height equal to that attained in 1882, namely, 44.8 on the Belmont gauge, the discharge would no doubt have been about 1 540 000.

At Helena.—At Helena, in 1882, the highest recorded discharge is 1 558 485, taken with the meter. There is an observation showing a discharge a little greater than this, taken a few days later, but it was made with rod floats, and the area was interpolated. The correction, by Francis's formula, if the rods were of the same length as those used at Columbus, would be 0.95, the proper correction, according to the curves of vertical velocity, being 0.93.

It is doubtful, then, from the record, whether so great a discharge as 1 550 000 passed. In 1893, at an average stage of 47.8, a mean discharge of 1 527 501 is recorded.* The stage reached in 1882 was only 47.2. There were some circumstances, however, that indicate the possibility of an increased discharge at Helena, even at a lower stage.

* So taken with meters at 0.6 depth. Discharges taken with double floats on the same days gave a little more.

Within 20 miles below Helena, there were several considerable crevasses, discharging some 150 000 cu. ft. per second, which must have lowered the water line a good deal, while giving an increased slope. It is perfectly possible, then, that the recorded discharge was genuine. Even in 1893, the levees of the White River basin extended only about 22 miles below Helena, and there was a gap of some 25 miles next succeeding. It is not to be doubted that an important influence was exerted on the Helena gauge by the depression of the water line below that station thus produced.

There is also a record of a discharge of about 1 310 000, at a stage of 45.7. That a discharge of 200 000 ft. more should be passed at an additional height of only 2 ft. indicates a great disturbance of the ordinary relations.

Influence of Saint Francis Basin.—The influence of the vast unleveed front of the Saint Francis Basin on the discharge and on the gauge height at Helena and at points below is indeed a question which is not well understood, and upon which opinions have been divided. It has been reasoned out on various lines. It was for a long time maintained by levee engineers, and by the author among others, that the effect of leaving the front of this great reservoir open was rather to increase the discharge and the height of the flood line at the foot of the basin. The argument in support of this view ran somewhat thus: Given a river inclosed by levees, on the one side of which is a great basin like that of the Saint Francis. Given a flood, the greatest that can be restrained by those levees. Then, near the height of the flood, the river-bed, thus enlarged artificially, is doing its whole duty, and carrying a discharge of 1 500 000 ft. Now let a breach be made in the levee bordering the reservoir. A volume, say of 300 000 cu. ft. per second is abstracted from the mass moving in the channel, and the discharge is reduced to 1 200 000. Then the stream is doing less than its duty. Now let the quantity of water lately abstracted be returned at the foot of the basin suddenly, as it is always returned, at a rate faster than that at which it was withdrawn. Then, as the river was lately doing less than its duty, so now it will have to do more, and the discharge at the foot of the basin will be greater than it would have been had the levees remained unbroken. So likewise the flood height will be increased by reason of the additional discharge. Not only so, but of the mass of water thus returned, a part has a low velocity and is projected into the

river at an unfavorable angle, so that a rise of the water surface will be produced until sufficient height is acquired to confer the requisite velocity.

This position was disputed some years ago by Captain Rossell, and has lately been attacked with much ability by Captain Townsend.* These engineers draw their arguments from the relations known to prevail between the different gauges. For instance, a comparison is made of the Cairo and Helena gauges, care being taken to select the crests of rises or the extreme low points of falls, so as to readily identify the corresponding points, and not be forced to apply a somewhat doubtful time interval. It is found that at all stages from zero to bank-full, there is, on the whole, a striking parallelism between the gauges, Helena being usually a little the higher. The relation is somewhat variable, depending mainly on the sharpness or duration of the rise, but if the latter be long enough to be propagated uniformly down the river, Helena will be about 0.5 ft. ahead of Cairo. At stages beyond bank-full, however, this relation ceases. Cairo then becomes the higher, and this by a constantly increasing quantity with the magnitude of the rise, so that in 1882 it was 4.7 ft. above Helena. In 1893 it was only 1.4 ft. above. This remarkable departure from previously existing relations is ascribed by Captain Townsend to the influence of the Saint Francis Basin acting as a reservoir. According to this view, the water abstracted from the upper end of the basin is retained in the reservoir a sufficient time to allow the crest of the flood to pass Helena before it is returned. When it is returned, it is not in sufficient quantity to raise the flood line to the height which it would have attained in a confined state.

This argument is very forcible, and it has the merit of being based on accurate and indisputable data. Discharge observations and measurements of flow over banks, through crevasses and across points, are attended with many elements of error and uncertainty. Gauge records, however, may be kept with any desirable degree of exactitude. There are no gaps in them. Errors, if there are any, are always subject to correction, and the relations between neighboring or even distant gauges maintain a considerable degree of consistency when allowance is made for disturbing causes, such as duration of rise, effect of tributaries, crevasses, etc.

* "Report of the Mississippi River Commission for 1894," p. 2967, *et seq.*

The plain consequence to which this reasoning leads is that if the front of the Saint Francis Basin were completely sealed by levees, the gauge at Helena, in long-continued floods, would eventually attain a height at least equal to that reached at Cairo.

Captain Townsend does not attribute the present diminished gauge height at Helena wholly to the Saint Francis Basin, but ascribes a part of it to the condition of the White River Basin. The facts

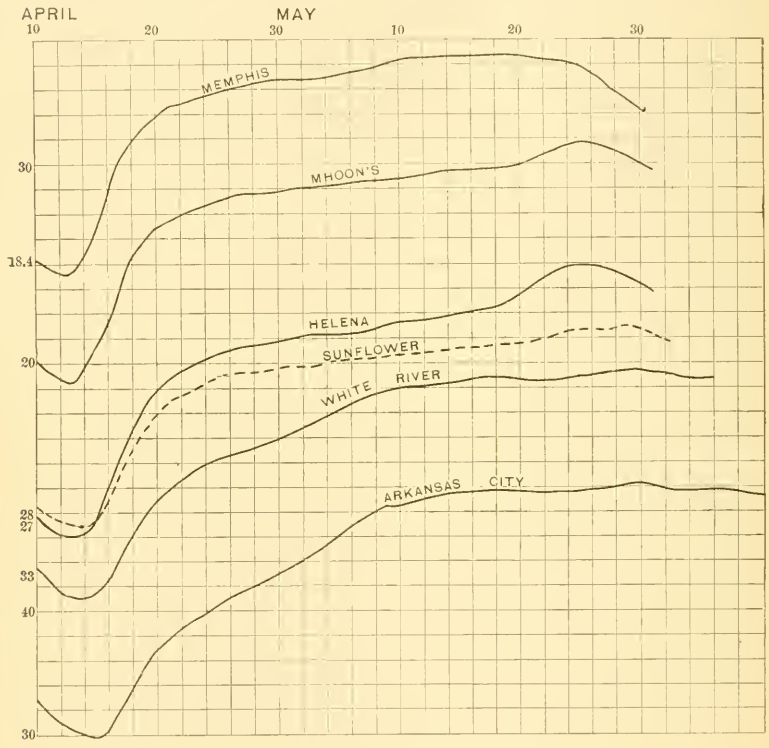


FIG. 30.

developed in the study of the Helena discharges point to the same conclusion. If the discharges can be relied on at all, the quantity of water passed at Helena in 1893 was about equal to that which passed Columbus and New Madrid, increased by the discharge of the Saint Francis River, which was at a low or medium stage. It was merely the height, then, and not the discharge, which was affected in this year.

Return of Flood Water.—The manner of the return of the flood water

from the Saint Francis Basin may best be studied by referring to the hydrographs in Fig. 30, for the high water of 1893 of Memphis, 76.5 miles above Helena, of Mhoon's Landing, 29.2 miles above Helena, of Sunflower Landing, 46.2 miles below, and of the mouth of White River, 91.7 miles below. It will be seen that the Helena gauge moved along, nearly parallel with that of Memphis, until the river reached its height at the latter place and had even begun to fall, when, about May 19th, at Helena, a rapid upward movement began at the rate of 0.5 ft. per day. This rate prevailed for about a week. The rise from May 17th to May 25th was 3.3 ft. The gauge at Mhoon's Landing sympathized with this movement, either because the return flow began so high up, or from the effect of the back water. From the appearance of the hydrograph it was probably the latter. The discharge increased, during the same time, from about 1 300 000 to more than 1 500 000. This is a greater rate of increase than is usual, even at such high stages, and points to extraordinary causes. The most prominent of these is the great increase of slope produced, first, by the sharp elevation of the water surface at Helena, and second, by the lowering of the water surface below Helena by the gap in the Arkansas levees acting as an outlet or waste weir. The amount of the increase of slope is very fairly deduced from the changes of general slope, as revealed by the comparison of the adjacent gauges below. Sunflower Landing, the next gauge station below, is directly opposite a part of the gap, and the relations between the Helena and the Sunflower gauges represent more closely than usual the local conditions. Assuming a fixed datum area of 220 000, the following table may be constructed:

Date.	Gauge.	Assumed area.	Assumed mean depth.	Slope, Helena-Sunflower	Assumed velocity.	Assumed theoretical velocity.
May 17...	44.6	203 526	40.8	0.0000738	6.45	5.49
25...	47.9	220 000	44.0	0.0000816	6.95	5 99

The coefficients were 117.5 and 116.5. The local slopes were measured, and are given as 0.0000648 and 0.0000836 respectively.

Height to be Attained at Helena.—Can the height which the gauge would have attained if there had been no disturbing influences above or below be deduced from the Helena discharges? If so, it can only be by extending to high stages the analogies developed at heights up to bank-full.

For this purpose the observations of 1890, 1891, 1892 and 1893 are of no use, for none of them were taken at stages below bank-full. The records of the years 1882 and 1884–85, however, cover all stages from low water up.

The observations of 1882 exhibit many discrepancies, as might have been expected. They were mostly made after the flood, so that it is impossible to take an ascending series of discharges, as in 1884–85 at Arkansas City. It is assumed that the descending series moved nearly parallel to the ascending, as usual; the law of the progression of the discharge will be found even from these imperfect data. The absolute quantity of the discharge, however, will not be obtained. It may be considerably more than that indicated by the descending series, as it would be less than that shown by the ascending series.

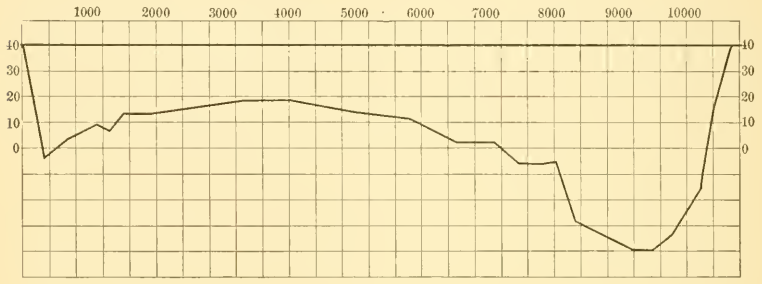


Fig. 31.

It was remarked in relation to the discharge at Columbus that the lower part of the curve followed a somewhat different law of progression from that which governed the upper part. This is found also to be the case at Helena and at Arkansas City. It is nothing more than might have been expected, when the shape of the cross-section of the river is considered, and especially of the wide sections which appear to control the height of the water. Fig. 31 represents the cross-section of the wide reach above Helena. Here, as at Arkansas City, at a stage about 20 ft. below bank-full, or at a reading of about 20 ft. on the gauge, the river spreads over an immense sand bar 1 500 ft. wide at the latter place and 4 000 ft. wide at Helena. Previously to this time it has flowed in a comparatively narrow and deep channel of nearly the same type as the discharge section. There is no reason why it should follow the same law after this change. On the contrary, the rate of

the discharge should increase, in consequence of the great width and shallow depth of the new part of the section, the relatively rapid augmentation of the velocity and the doubly rapid increase of area. The rate of increase, however, will be slow at first, on account of the small velocity of the new section for the first few feet after it has been covered. If these things be so, then the discharge curve in such localities will really be a compound curve.

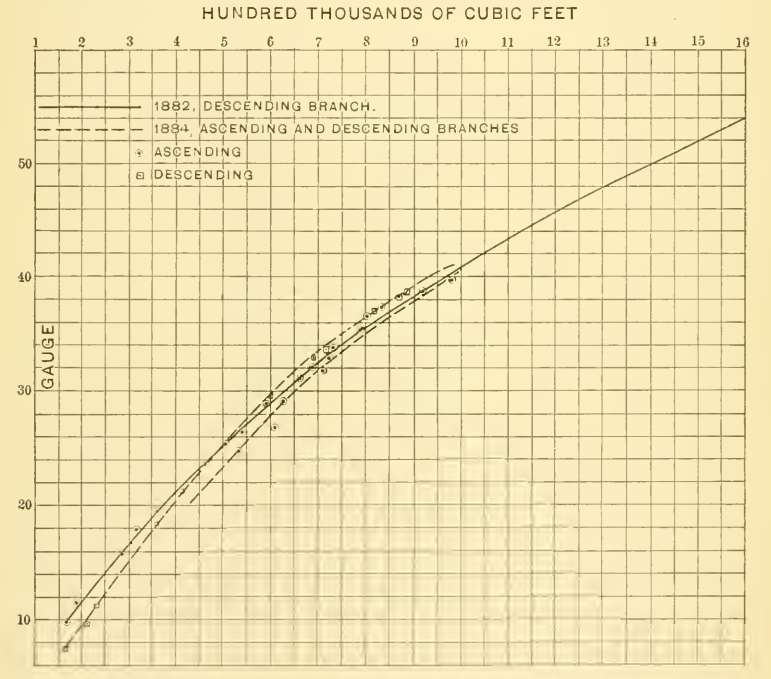


FIG. 32.

Helena Discharge of 1882.—In the construction of the following table, the observations made at nearly similar stages have been combined and the means taken. For instance, all the observations taken between 38 and 39 ft. on a falling river, eight in number, have been used for the figure of 916 500, given in the plot as corresponding to the mean stage of 38.6. As it is the object to trace the descending branch of the curve, none of the measurements made during the progress of a rise have been used The table shows the progression of the discharge in the descending branch from low water to bank-full, and the extension of the same

relations to stages higher than have ever been reached in a confined river. The principal break in the curve seems to occur at about 30 ft. The observations of 1884–85 also have been plotted and are given in Fig. 32. It will be seen that at low stages they depart somewhat from the curve of 1882, while at high stages they exhibit nearly the same law of progression. It seems hardly worth while to tabulate them.

Gauge.	Discharge.	First Difference.	Second Difference.
10	171 000	.....	.....
12	208 000	37 000	.....
14	247 000	39 000	2 000
16	288 000	41 000	.....
18	331 000	43 000	.....
20	376 000	45 000	.....
22	423 000	47 000	.....
24	472 000	49 000	.....
26	523 000	51 000	.....
28	576 000	53 000	.....
30	631 000	55 000	.....
32	690 000	59 000	4 000
34	753 000	63 000	.....
36	820 000	67 000	.....
38	891 000	71 000	.....
40	966 000	75 000	.....
42	1 045 000	79 000	.....
44	1 128 000	83 000	.....
46	1 215 000	87 000	.....
48	1 306 000	91 000	.....
50	1 401 000	95 000	.....
52	1 500 000	99 000	.....
54	1 603 000	103 000	.....

In 1882, at stages above 40, there is exhibited a manifest tendency to depart still further from the law of progression which the river seems to have observed while within its banks. At 41.6 on the gauge, a mean of 12 consecutive observations gives a discharge of 1 101 000 cu. ft., which is considerably in excess of what it should have been with the relations prevailing at that time. Probably this is attributable to an escape of water below Helena. These measurements were taken after the flood, and there were several deep crevasses still running at this stage which no doubt operated to lower the water surface at the latter point. This, indeed, is clearly indicated by the gauge at Malone's Landing, 49.2 miles below, which was nearly 2 ft. lower than its ordinary relation to the Helena gauge would prescribe.

It is not safe to reason from measurements taken at a stage of 30-40 ft. to stages of 50 ft. or more, without some facts to check up on. From the nature of the case, there are no high-water discharges at Helena which are not affected by the very causes which it is sought to eliminate. Reasoning from such data as are available, if it is assumed

that at about 40.5, in 1885, a discharge of 1 015 000 passed as recorded, then by the rule of progression which seems to prevail, there should be a discharge of about 1 430 000 at a stage of 50. By the same reasoning a discharge of 1 500 000 should pass at a gauge reading of about 51.4. It is recorded that such a discharge passed in 1893 at 47.9, indicating, so far as this argument may be relied on, that the flood line had been depressed by high-water influences 3.5 ft. So far, then, Captain Townsend's conclusions are confirmed, and the confinement of the river by the closure of the Saint Francis and White River Basins will have the effect of augmenting the gauge height at Helena.

Will this increased flood-height be propagated down the river, and, if so, how far and to what extent? The answer to this question will depend, in part at least, on the importance of the White River Basin as an agent in affecting the water plane.

Influence of White River Basin.—Even at the date of this writing, there is a gap in the line of levees of the White River Basin some 10 miles in length, which, however, will be closed, no doubt, in two or three years. This gap is sufficient to discharge 200 000 or 300 000 cu. ft. per second at high water, and to depress the flood plane in its vicinity 3 ft. or so.* Such a steepening of the slope at Helena must have a very considerable influence on the velocity at the latter point, and consequently on the flood height. It is not impossible that a great part of the effect usually attributed to the Saint Francis Basin may be more properly due to the outlets below Helena. It is well known that the outlets below Arkansas City in 1886 and 1887 had a notable effect in lowering the high-water line at that place. In 1887 a discharge measured as a mean of 1 440 000 cu. ft. passed Arkansas City at a height of 46.6 ft. Such a discharge corresponds normally to a height of nearly 50 ft.

The water which is poured over the bank through the gaps into the White River Basin soon returns to the Mississippi, usually through the channel of White River itself, if that stream be not very high. If it be in a state of extreme flood, a part of its discharge is transferred to the foot of the basin by overflow, and enters the Mississippi along the line of the Cypress Creek levee. As the basin is only one-sixth the size of the Saint Francis Basin, its effect is small com-

* Captain Roessler estimates the "potential high-water line," or water line as it would be if the stream were entirely confined, as 2.7 ft., above the actual flood line in some localities.

pared to that immense territory, though still of great importance in several respects.

It was seen that the discharge at Columbus in 1882 was probably 1 600 000 cu. ft. or more, including the volume which passed in the swamp. Now the discharge which passed Helena was only 1 500 000 ft. or so. Does not this indicate that a part of the water was held back in the Saint Francis Basin? It does not. The levees of the upper part of the Yazoo Basin, from the hills below Memphis to Glendale, opposite Helena, were in a ruinous condition, and the volume of water which poured over them was estimated in 1882, and again under circumstances almost precisely similar in 1883, at 290 000 cu. ft. per second. As has been stated, these measurements are always excessive; but allowing only half of the above estimate, it will account, not only for the 100 000 cu. ft. of difference between Columbus and Helena, but also for the discharge of the Saint Francis River proper, estimated at about 50 000 cu. ft.

What Height Could be Reached at Cairo?—If it be true, as seems probable, that in a confined river, in prolonged floods, the river would rise to a height on the Helena gauge equal to or greater than that attained at Cairo, what would now be the altitude which that standard of the river would reach? For even at Cairo the river is not closely restrained. There is a considerable flow over the bank at Bird's Point, in Missouri, just opposite, which was measured in 1883 and reported at 216 000 cu. ft. per second. It may reasonably be supposed that such a loss of volume as this would have a material effect on the water line, and indeed it has been maintained that it would depress it 2 ft. or more.

Water Escape Below Cairo.—Now, in 1883 the river attained a higher point at Cairo than in 1882 or in any other year, before or since. The records for the three remarkable seasons of 1882, 1883 and 1884, being the means for six successive days, are 51.55 ft., 52.03 ft. and 51.72 ft. The measurement of the flow across Bird's Point in 1883 was taken at a stage of 52.08 ft., so it fairly represents the maximum. Of this volume 111 000 cu. ft. per second were returned within 8 miles below Cairo, and 37 000 cu. ft. more at about 13 or 14 miles below; but there was reported a great outflow in the bend below the latter point, estimated at more than 300 000 cu. ft., most of which was returned again about Columbus; but the records give a large net escape until New

Madrid was reached, when it all or nearly all came back into the main river.*

Discharge Across "Points."—The discharge of water across "points," or peninsulas unprotected by levees, and the return of the same water in the bends below, sometimes reaches enormous proportions. These great movements have not often been carefully estimated. They were roughly measured in 1883, in the hydrographic reconnaissance of the Saint Francis front for the purpose of estimating the water escape of that year, and they seem sometimes to have attained the astonishing figures of 300 000 or 400 000 cu. ft. per second. In 1893, the discharges across Ashbrook's Point and Carter's Point were measured by Captain Townsend and found to be (at the highest stage) about 200 000 and 300 000 cu. ft. respectively.

Disturbances of Level.—It would seem that the transference of these enormous masses of water could not take place without serious disturbances of level. The abstraction of 200 000 cu. ft. of water per second permanently would reduce the water level at the point where it occurred at least 2 or 3 ft., and the injection into a stream of as much water from a tributary should raise it by a like amount. It is easily seen, however, that where the outlet and the return inlet are only 10 or 12 miles apart, such processes are incompatible with one another. The whole fall of the river in 14 miles, in the neighborhood of Carter's Point, is only about 4 ft., and the river would thus be reduced to an absolute pond. No such phenomenon occurs. The slope is disturbed indeed, but not to any such extent, which would involve an impossibility. In 1893 the difference of water level between the upper and the lower sides of Carter's Point at high water, when the movement of 300 000 cu. ft. per second took place across the peninsula, was 4.4 ft. in a distance of 15.5 miles, or 0.284 ft. to the mile, a good deal less than the normal for the whole reach, which is 0.34, and corresponding to a disturbance of level of 0.85 for the 15.5 miles; provided, indeed it could be proved that the normal relation would have existed in this particular portion of the reach. This cannot be assumed without direct evidence. In a smaller flood, 1891, however, the relation was 5.3 ft. for the same distance, or a slope of 0.342 to the mile, namely, about the normal. There was a considerable discharge across the point even in 1891. It may be concluded that the difference of level intro-

* "Report of the Mississippi River Commission for 1884," pp. 2623 *et seq.*

duced by the combined influence of the outflow and inflow was about 1 ft. By the time Greenville was reached, 4 miles below, the slope had been restored and even a little more, the fall in the 4 miles being 2.4 ft.

Back Water.—The phenomenon of the rising of the water surface on the lower side of overflowed "points" in great as compared with small floods has been observed by levee engineers, and allowance is or should be made for it in establishing grades. The means by which the slope is so adjusted as to carry the discharge is what is commonly known as "back water"; that is, the effect of the ingurgitation into the stream on the lower side is to raise the water surface, and this immediately acts to check the velocity above, and hence to increase the cross-section, and this to a considerable distance. Without discussing the general question of back water, which has been made the subject of a good deal of investigation, it will suffice to look at the examples presented by recent floods in the Mississippi. One example has been given already on page 471, in the effect of the return flow at Helena upon the gauge at Mhoon's Landing, 29.7 miles above. Still more striking illustrations were afforded in 1882 and 1884 by the great return flow at Vicksburg from the Yazoo Basin, then only partially leveed. The effect of this tremendous engorgement was very perceptible at Hays's Landing, 45.6 miles above Vicksburg, as will be evident by an inspection of the hydrographs, given in Fig. 33. From the Greenville hydrograph, it is plain that the river would have been falling at Hays's Landing on March 18th, 1884, had it not been for the return wave at Vicksburg, the influence of which not only kept it from falling, but raised it about 1 ft. The curve of 1882 was somewhat affected by local conditions. Especially, the sudden fall at Hays's Landing on March 21st and at Vicksburg on March 22d was due to several great crevasses which occurred on the former day, some

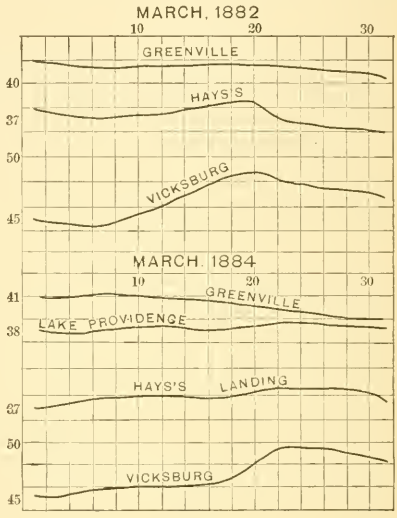


FIG. 33.

10 miles below Hays's. As the gauges at Greenville and at Hays's generally move nearly parallel, it is to be inferred that, instead of rising 1 ft. in 1884, the river would have fallen about 1.5 ft. up to March 25th, had it not been for the back water. The effect of the latter has been, then, to raise the water surface at Hays's about 2.5 ft.

The relation between the Vicksburg gauge and that at Greenville is not so well established. It is affected by the stage of the Yazoo River and by other causes, but it is safe to infer that the gauge reading at Vicksburg, from the discharge which passed Greenville in the channel, would not have exceeded 44 ft. There was a rise, therefore, from the return flow of the Yazoo Basin of at least 5 ft., and half of this effect was produced 45 miles up the river. An effect of 2 ft. was produced at Lake Providence, 57 miles above Vicksburg, and it is thought to have been distinctly perceptible as high as Longwood, in Mississippi, 98 miles above Vicksburg. It should be observed, however, that the greater part of the return pour was discharged, not exactly at Vicksburg, but near the mouth of the Yazoo, 10 miles above.

Effect of Outlets and Inlets Below Cairo.—The effect produced at Cairo by the flow across Bird's Point for a few days would not have been much had the water restored to the channel remained there. As it was, the great outlets in Lucas Bend, 15 miles below, the water from which was partially returned near Columbus, 21 miles below Cairo, and the most of the remainder 40 to 45 miles below Cairo, must have in some measure offset the return below Bird's Point.

As to the quantity discharged over the latter point, there is room for much doubt. The same observer, in 1882, at a stage 4.5 ft. less, found the bank of Bird's Point exposed, and no escape at all from Commerce, in Missouri, to a point 14 miles below Cairo.* It seems hardly credible, therefore, that the Point should have been overflowed for a distance of 2 or 3 miles, to a depth varying from 9.5 to 11 ft., even at the highest stage of 1883. From the very crude methods employed, the results can hardly be accepted even as approximations.

The slope Cairo-New Madrid in 1882 was 0.369 to the mile; in 1883, 0.370; in 1880 (gauge at Cairo, 44.60 ft.), 0.358; in 1893, 0.358. Therefore the gauge height at Cairo in 1882 was not diminished below its usual relation, but rather increased. In 1882 the New Madrid

* "Report of the Mississippi River Commission for 1884," page 2597.

gauge * stood at 41.49 ft., while that at Cairo was 51.87 ft., a difference of 10.38 ft. In 1883 the gauges were 52.17 ft. and 41.74 ft., a difference of 10.43 ft. In 1884 the figures were 51.79 ft. —41.50 ft. = 10.29 ft.; in 1880 they were 44.60 ft. — 34.70 ft. = 9.90 ft.; in 1886, 50.98 ft. —40.38 ft. = 10.6 ft.; 1892, 48.3 ft. — 37.7 ft. = 10.6 ft.; in 1893, 49.3 ft. — 38.7 ft. = 10.6 ft. New Madrid is chosen as the standard of comparison because all or almost all of the water which enters the Saint Francis Basin below Commerce returns by the New Madrid ridge and passes in the channel at New Madrid. The relation between the Cairo and the New Madrid gauges is shown in Fig. 34. The difference between them increases with height, but at high stages it seems nearly uniform. In 1892 and 1893 there could have been little or no escape at Cairo; and the fact that the same relations prevailed between the two gauges in the three great floods of 1882, 1883 and 1884 as in 1892 and 1893 leads to the inference that there was no extraordinary disturbance at Cairo, unless indeed there was a similar disturbance at New Madrid. Now, the loss of volume below New Madrid, either in 1882 or 1883, was not considerable. There was no great outflow, even as shown by the observations of 1883, for about 50 miles below that station. There is reported, indeed, a great inflow just above it; but without a corresponding outlet below, there should have been no depression of the water line at New Madrid. A difference of 11 ft. between the Cairo and New Madrid gauges is probably not far from the truth. This relation would give for a confined river, in 1882, a gauge height at Cairo of 52.5 ft.

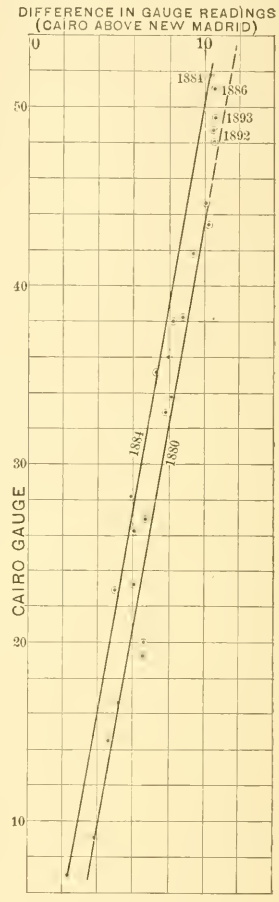


FIG. 34.

* Deduced from the high-water observations of 1882. The gauge here mentioned is, in strictness, the Morrison's Landing gauge, which is 1.3 miles above New Madrid. Lately (1893) a new gauge has been set up at New Madrid proper, and the Morrison's Landing gauge discontinued; 38.7 on the latter corresponds to 38.1 on the former.

Discrepancies in Estimates of Escapes Over Banks.—The estimates of flow over the banks in 1883 seem excessive from another point of view. It was seen that in 1882 the quantity of water reported as passing at Columbus outside of the channel was about 86 000 cu. ft. In 1883 it is reported at 300 000 cu. ft. or more. The difference in the mean gauge readings at Cairo was 0.5 ft. The reading at New Madrid was 0.25 higher. Very nearly the same conditions prevailed in the Yazoo Basin and the Saint Francis Basin. The gauge at Helena was lower by 0.3 ft. Therefore the discharge could not have been considerably greater in 1883. Therefore either the estimate of 1882 was too small or that of 1883 was too great. By taking the discharge observations at New Madrid in 1893, it will be found that a stage of 41.5 ft. on the Morrison's Landing gauge (40.9 ft. on the present New Madrid gauge) corresponds to a discharge of about 1 600 000 cu. ft. It is therefore fair to infer that this was about the actual discharge which passed Cairo in 1882, and that the gauge height at the latter point, if the river had been confined, would not have exceeded 52.5 ft.

Relations between Helena and Arkansas City.—At Helena, according to Captain Townsend's reasoning the height would have been a little greater, about 53.0 ft. From the estimate given on page 475, a very similar conclusion is drawn. Now, what will this correspond to at Arkansas City, with the White and Arkansas Rivers at low and at high stages respectively?

It is very difficult to eliminate from the evidence the effect of the discharge from the tributaries. Taking only well defined and prolonged flood waves, and using the averages for several days, it will be found that for low stages of the White and Arkansas Rivers, the Arkansas City gauge usually stands about 0.4 ft. or 0.5 ft. above Cairo, or in fact nearly equal to the Helena gauge. When the tributaries are in great flood, Arkansas City has been known to be as much as 4.7 ft. the higher. At the same time, it was 3.9 ft. higher than Helena. This was in May, 1885, when the Mississippi was at a stage less than bank-full. Generally, at high stages, but with a confined river, it may be said that, with equal discharges, Arkansas City stands a little higher than Cairo. In 1893, for instance, with a discharge of about 1 450 000 cu. ft. at Cairo, the gauge stood at about 49.25 ft. for seven days. At Arkansas City, for a like discharge, it stood at 50.0 ft.

Effect of Floods from the White and Arkansas Rivers.—Now, what would be the effect upon the gauge at the latter place of a great flood from

the two tributaries—for they nearly always come out together? As a general rule, the upper waters of the Arkansas River contribute little to the Mississippi, flowing through an arid and sandy region. It is from the abundant rainfall upon the southern slope of the Ozark Mountains that the damaging floods derive their origin, or at least receive their principal increment. The White River drains the northern slope of the same range; and the storms which produce a freshet in one bring out the other also, as well as, ordinarily, the Saint Francis. The flood of May, 1885, just mentioned, reached 28.60 ft. at Little Rock (engineers' gauge)* and 25.60 ft. at Jacksonport. This should have corresponded to a discharge, as measured, of about 330 000 cu. ft. for the two rivers. This and the increment from the Saint Francis, whatever it may have been, raised the gauge at Arkansas City about 4.7 ft. The greatest flood which has occurred since the time of accurate observation, and one of the greatest ever known in these rivers, took place in May, 1892. This flood reached 31.2 ft. at Little Rock, and 30.4 ft. at Jacksonport. The measured discharge at the former place is given as 456 000 cu. ft., but this is believed to have been excessive. Judging from the observations of 1879 and of 1893, it should have been, for an average of several days, more than 300 000 cu. ft. In White River it was about 170 000 cu. ft. The result of this addition to the discharge of the main river should have been an increase of height (reasoning from the experience of 1885) of about 6 ft. at Arkansas City over Cairo. The reading at the latter station was, for a mean of six days, 48.0 ft. The height actually recorded at Arkansas City was 50 ft. There was a loss of water estimated at 300 000 cu. ft. per second around the end of the levee above Arkansas City, and there was a considerable crevasse 14 miles below that place, discharging, it was said, about 80 000 cu. ft. In 1893, the discharge from the tributaries, at the height of the flood, may be estimated at 100 000 cu. ft. The loss around the end of the Cypress Creek levee is given as 97 000 cu. ft. Consequently the gauge stood at nearly its normal relation to Cairo.

In 1882, at the crest of the flood, the White River was discharging about 90 000 cu. ft. and the Arkansas a little less.

At High Water, the Tributaries Discharge Partly Through the Basin.— It must not be forgotten that the two great tributaries which empty above Arkansas City do not pour the whole of their discharge at high

* There are no less than three gauges at Little Rock, and they are frequently confused.

water directly into the Mississippi. At such times the bank is overflowed to the depth of several feet, in some places 15 ft. or 20 ft., and the entire White River Basin is converted into a lake of 1 000 square miles area, or, including portions of the valleys of the two rivers within the reach of back water, of 1 200 square miles. Through this lake a part of the discharge of the two rivers pours, the greater portion through the mouth of White River, the rest through the old mouth of the Arkansas. A considerable fraction of the combined river discharge and overflow water, however, goes to swell the volume of the lake, and is carried with a gentle current to the foot of the basin, a few miles above Arkansas City, where it is finally emptied into the main river. The lake partially acts as a regulating reservoir, and sharp discharge waves are modified by it.

Effect on Gauge Height of Discharge of Tributaries.—Why a discharge of the tributaries of 330 000 cu. ft. in May, 1885, should produce an increase of height at Arkansas City over Helena of only 3.9 ft. remains to be examined. It has been seen that at high stages about 50 000 cu. ft. increase of discharge corresponds to 1 ft. increase of height. An addition of 330 000 cu. ft., therefore, should produce a difference between Helena and Arkansas City of 6.6 ft.

It has long been known that the discharge of a stream after the reception of a tributary is less than the combined discharges of the two. This is principally ascribable to two causes. First, the freshets from the tributaries do not reach the main stream at the crest of its own principal rise; and second, the channels, and, still more, the flood plains of the minor streams, where such exist, have a considerable reservoir capacity, by which the excessive discharges of the latter, always comparatively brief, are held back and distributed over several days.

The most formidable freshets of the White and Arkansas Rivers, as has been stated, owe their origin to the rain which falls on the northern and southern slopes of the Ozark Mountains and their outliers. These rains are sometimes very heavy. In May, 1892, at one station in the Arkansas Valley, a fall for the month of 18.5 ins. was reported, and the general depth of rain on both slopes of the mountains was about 12 ins.

The cyclonic storms from which this precipitation proceeds now continue on their way to the upper waters. They drench the valley of the Ohio in spring or of the Missouri and upper Mississippi in summer, one or two days after they have done their work in Arkansas.

Now the floods of the lower tributaries reach their mouth in three or four days. Those of the upper rivers do not get to the same point for nine or ten days ; or if they have to pass through the Saint Francis Basin, not for three weeks. Consequently the freshets from the White and Arkansas have done their worst before the main rise comes, and when it reaches their mouth, it finds them falling.

In 1885, the Arkansas attained its maximum at Little Rock on April 27th to 29th; the White River at Jacksonport on April 28th. The Mississippi reached its height at Helena on May 4th, and the crest of the rise reached the joint mouth of the two tributaries about May 6th. By that time, the minor rivers had fallen several feet, and their joint discharge probably did not exceed the 200 000 cu. ft. accounted for by the rise.

Unfortunately, no discharges were taken at Arkansas City during the second flood of 1885, in May, but reasoning from the measurements made during the previous rise in January, the masses of water which passed the two places did not differ by more than the above quantity. The discharge at Helena, at the climax of the May flood, at the maximum stage of 38.75 ft., seems to have been about 950 000 cu. ft. ; and the Arkansas City discharge, four days later, at a height of 42.60 ft., should have been about 1 140 000 cu. ft. The difference between these quantities, or 190 000 cu. ft., should represent, within pretty narrow limits, the addition to the volume of the flood between Helena and Arkansas City.

In 1892, the Arkansas reached the tremendous height of 31.2 ft. at Little Rock on May 21st. The White River touched 30.4 ft. at Jacksonport. Now the Mississippi had passed its maximum at Helena by three weeks when this storm came, and had fallen 1.3 ft.; and the subsequent rise, which was due to the storm, reached the mouth of White River about June 16th, when it encountered the end of another rise, also of very formidable proportions.

But though it should happen that the crest of the rise of the main river should meet a tributary while at its greatest height, yet the effect upon the Mississippi will not be equal to that due to the maximum discharge of the tributary, unless the latter keep up for a month or so.

Topography of White River Basin.—What is commonly known as the White River Basin does not represent the whole reservoir surface available for the impounding of the waters of the two tributaries while in flood. The White River itself has a wide bottom, averaging,

perhaps, 4 or 5 miles in breadth, and it overflows its banks very readily. At Clarendon, it overflows at a stage of about 24 ft., some 10 ft. below high water. The Arkansas has a still wider and deeper bottom from Pine Bluff down, some 10 miles in breadth. The combined area of the basin proper and these outliers is about 1 160 square miles. When the Mississippi is high, the entire basin is filled, either from the main river or the tributaries, or both, the back water extending a long distance up both the minor streams. When the Mississippi is about bank-full and the tributaries very high, the latter still overflow their banks, especially White River, which is a stream of flat slope in its lower part. Of the main basin some parts are very low, far below mean high water or bank-full of the Mississippi, and such regions are overflowed in all considerable freshets of the subsidiary streams, which usually coincide with tolerably or exceedingly high stages of the Mississippi.

Effect of the Basin as a Reservoir.—If the maximum discharge of the Arkansas and White Rivers combined for several days is assumed to be 500 000 cu. ft., and if this volume is supposed to be poured into the White River Basin, already overflowed from the main river, it will inundate it in one day to the depth of 1.33 ft. If the discharge of the Mississippi is assumed to be increased at the rate of 50 000 cu. ft. for each foot of rise, the rise of 1.33 ft. will increase the discharge by about 66 500 cu. ft. in the first 24 hours, or for the average of the day, 33 250 cu. ft. This increased discharge will have the effect of lowering the water in the basin about 0.09 ft. The net rise in the basin in the first 24 hours will be 1.24 ft.

On the second day there will enter the basin 500 000 cu. ft. per second as before, but there will leave the basin 62 000 cu. ft., making the net gain 438 000 cu. ft. This will raise the basin about 1.17 ft. more; but this rise will cause an additional discharge of 58 500 cu. ft., or for the average of the second day, 29 250 cu. ft. This will cause a fall of about 0.08 ft.; net rise of the basin, 1.09 ft.; total rise, 2.33 ft. Third day, net discharge into the basin, 383 500 cu. ft.; rise, 1.02 ft.; additional mean discharge, 25 000 cu. ft.; consequent fall, 0.07 ft.; net rise, 0.95 ft.; total rise, 3.28 ft. A continuation of this process gives for the fourth day: Total rise, 4.11 ft.; fifth day, 4.88 ft.; sixth day, 5.52 ft.; seventh day, 6.06 ft. On the ninth day the rise of the basin would be about 7 ft.; on the twelfth about 8 ft.; on the seventeenth about 9 ft.; in a month, about 9.8 ft.

The freshets of the White and Arkansas Rivers do not remain long at an excessive height. In the great flood of 1892, the Arkansas remained at an extreme stage of 30 ft. or more at Little Rock for five days. Its extreme discharge of 400 000 cu. ft., or whatever it was, was for only one day. The stage of 30 ft. at Little Rock seems to correspond to a discharge of about 300 000 cu. ft. The water fell in two more days to 28 ft., and in two more to 26 ft., indicating discharges of 260 000 cu. ft. and 220 000 cu. ft. respectively. At Clarendon, the high stage was maintained with greater persistency from the effect of back water, but the discharge decreased from the falling off of the supply from above, and, consequently, the diminished slope.

Now, the rapid decrease of the supply into the reservoir very soon equalizes the inflow and the outflow, and presently the latter becomes the greater.

It must not be forgotten, however, that the tributaries sometimes remain for weeks together at stages much greater than the average. In 1890, White River remained for four months at an average stage of about 26 ft. at Jacksonport, and was never below 21 ft. In such cases the actual discharge produces its full effect, or nearly so.

Estimation of Maximum Discharge.—In estimating, then, the maximum discharge which has passed down the Mississippi in a given year at Helena, it will not be proper to take the maximum discharge at Cairo and add to it the maximum discharge of the Saint Francis; and for Arkansas City, to increase this by the maximum discharge of the White and Arkansas. Such a process, in 1882, resulted in absurd overestimates; and, indeed, the discharge of that year has always been exaggerated. As it is a matter of great consequence to obtain correct information on this point, a revision of the figures which have been presented will be made.

At Columbus.—A probable value for the Columbus discharge was found from the observations of that year to be 1 573 000 cu. ft. To this quantity a correction of 2% should be applied for the defective formula, bringing the net discharge to about 1 540 000 cu. ft. This corresponds with the results of the Columbus and New Madrid observations of 1893, quoted on page 467. Besides this, there was an escape over the banks, estimated at 86 000 cu. ft. Reasons have been given for doubting or altogether discrediting these estimates, and the calculation on page 481, made from the New Madrid measurements of 1893, of 1 600 000 cu. ft. for the total discharge of 1882 at Columbus, including what

passed outside of the channel, seems entirely reasonable. It may be affirmed that the measurements of 1893, checked at two points, made with the best instruments, by experienced men, and with the precautions shown to be necessary, and agreeing well together, taken, moreover, near extreme flood, are safe and trustworthy sources of information.

At Arkansas City.—The White River was at a stage of 21.8 ft. at Jacksonport when the crest of the rise passed the mouths of the two rivers; the Arkansas marked 15.1 ft. at Little Rock, and both were stationary, after having been much higher. It is likely, therefore, that they contributed nearly their full discharge, which was probably about 140 000 cu. ft. If 50 000 cu. ft. is added for the discharge of the Saint Francis, there will be about 1 790 000 cu. ft. for the volume which would have passed Arkansas City in a second in 1882 in a confined river. This estimate, however, is subject to deductions from several causes.

Deductions from Above Estimate.—The channel and flood plain of the river afford a considerable reservoir surface, to fill which will consume a portion of the discharge. This effect is particularly marked, as has been observed by Humphreys and Abbot,* toward the upper part of the alluvial valley, where the oscillations of height at the several gauge stations are sharper and more frequent. The effect, however, is cumulative, and the diminution of discharge continues as the river is descended. The extent of the reservoir influence will depend mainly on the duration of the rise. It is believed that an undue importance was attached to it by Humphreys and Abbot, who, reasoning from the experience of a short rise in 1858, estimated a regular loss of 140 000 cu. ft. of discharge between Cairo and Helena. Such a loss may occur where a heavy and sharp flood comes out on a low river and is succeeded by a rapid decline, but it cannot take place when the channel is already full and when the highest stage of the rise is prolonged for several days. Even under these circumstances, however, the reservoir influence is very perceptible. At Columbus, in 1882, the high stage of 102.8 ft. lasted only for a single day. In four days more the river had fallen to 102.1 ft., and the discharge presumably from 1 600 000 to 1 560 000 cu. ft. Now it takes about four days in a confined river for the crest of the rise to pass from Columbus to Helena. Taking the case of a confined river, and assuming the distance between levees when completed to be the same as already exists between Arkansas City and Wilson's

* "Report on the Physics and Hydraulics of the Mississippi River," p. 371.

Point, namely, about 2.5 miles, the reservoir space at high water between Columbus and Helena will be about 712 square miles, or about 19 850 000 000 sq. ft. The rise at Helena in four days was about 1 ft. The river at Columbus will be supposed to be stationary (it varied from 102.62 ft. to 102.79 ft.). To fill the basin an average of 0.5 ft. in one day would take about 10 000 000 000 cu. ft., or in four days about 2 500 000 000 cu. ft. This is equivalent to about 28 900 cu. ft. per second. This much of the discharge, therefore, will be consumed in filling the reservoir during the process of raising the river at Helena 1 ft., and the discharge which passes Helena will be less than the discharge which has passed Columbus by so much. On the fifth day, the discharge will be reduced by the fall from above by about the same amount (from about 1 600 000 to 1 571 500 cu. ft.), and the next day by as much more. As an approximation, the maximum discharge passing Helena in 1882, in a confined river, would have been nearly 30 000 cu. ft. per second less than passed Columbus. At Arkansas City, by the same mode of reasoning, the loss would have been increased to about 42 000 cu. ft., at Wilson's Point to 52 000 cu. ft.

Experience with confined waters at time of extreme flood is very limited. That condition is usually soon disturbed by crevasses and other complications. It is a time of great activity, anxiety and worry, and accurate observations of the behavior of the river are scarce. With such unknown conditions, it is possible that unexpected phenomena may be developed, causing an increased discharge to pass at very high stages. From causes which are already known, it is likely that flood heights at such stages will be diminished still further below the estimates given.

First.—No account has been taken of the discharge over banks or between the margin of the river on either side and the adjacent levee. The average distance between levees, as has already been remarked, from Arkansas City and Wilson's Point, is about 2.5 miles, of which space the river occupies nearly one-third. The levees are something like 12 ft. high. As the flood water gets deeper over these "foreshores," its velocity will increase rapidly. Even now, the "over-bank discharge" is an element which must be taken into account in every set of high-water measurements. At Arkansas City, in 1893, at the highest stage, this was estimated at 9 000 cu. ft., and at Wilson's Point it was over 30 000 cu. ft. Of course, the influence of this element will not be confined to the local space at each point that may lie between the

levee and the river, but will depend on the general section or the controlling section, whatever it may be.

Second.—At high stages the river flows between "water banks," as is often expressively said, for the upper part of its section, namely, that which is over bank-full height. Thus the friction and local disturbance ought to be somewhat diminished. It is not easy to estimate accurately the value of this element, but it would probably be not very great. In the ordinary formula, the substitution of banks of water for banks of earth for the upper 10 ft. of the section, even supposing the "water banks" to exert no resistance at all, would make a difference of only about 0.02 ft. per second in the velocity, or say 5 000 cu. ft. in the discharge.

Third.—It has been seen that it is possible for serious permanent alterations to occur in the bed of the river, and that the slope also undergoes modifications. This simply means that the river is relatively lower at some points than it used to be, as compared with the standard, Arkansas City. The confinement of the river, more or less complete, might be expected, indeed, to produce some effect on the bed. If a deterioration of the latter was brought about by the loss of volume over the banks, then it might reasonably be hoped that the retention of the water formerly lost by dispersion would have the effect of lowering the bottom in those places, so as to conform more nearly to the high-water slope.

Very sanguine expectations have been entertained of the scouring action of the powerful high-water forces thus brought into play. As the river never has been completely restrained, it cannot be said that the experiment has yet been fairly tried. The enlargement of cross-section at Wilson's Point, previously noticed, seems to have occurred in one season, from 1890 to 1891, and it has changed but little since that time. The relation between the gauges at Arkansas City and Lake Providence appears to have undergone no considerable alteration since 1885. The section at Arkansas City, with all its temporary mutations, remains the same. The discharge does not pass at a materially lower height than might have been predicted from the experience of previous years. So far, in the part of the river considered, the hopes entertained have not been realized.

Revised Estimate.—For all these causes, it seems reasonable to suppose that the extreme discharge of 1 790 000 cu. ft. found for Arkansas City in 1882 would have been diminished by at least 55 000 cu. ft.,

perhaps more, and that the maximum actual discharge which would have passed in a confined stream in that year would have been not greater than 1 735 000 cu. ft. This would require, by the methods previously used, a gauge height of about 55.8 ft.

It will be well to compare the floods of more recent years, and see whether the high water of 1882 was really the greatest of which there is any record, and consequently whether it was correctly adopted as a standard.

In 1883, the river attained its greatest recorded height at Cairo. There was a considerable flood in the Arkansas and White Rivers, but it did not coincide with the crest of the main rise. The river below Arkansas City was at an unusually depressed stage when the rise began; so the reservoir capacity of the channel was very great. Consequently the flood wave flattened out, and the gauge at Vicksburg was 5 ft. lower than in 1882.

The flood of 1884 was one of the greatest ever known. It may fairly be compared with 1882. The gauge at Cairo and at Helena was nearly the same as in 1882. At Vicksburg it was 0.15 ft. higher. No material change had occurred in the situation. No discharges were taken in 1884, and it is not easy to institute minute comparisons between them.

In 1886, there was a short and sharp rise which reached 51 ft. at Cairo and 48.1 ft. at Helena, the levees of the upper Yazoo District having been recently rebuilt. There were no serious freshets from the White and Arkansas Rivers, and the river at Vicksburg was 4.6 ft. lower than in 1882.

There was a great flood in 1890. It was not very high at Cairo, only 48.8 ft., but it was very long and persistent, and it was accompanied by an unparalleled stage of White River, which stood for four months at a mean height of 26 ft. at Jacksonport, the highest being 33.2 ft. (the greatest on record), and the lowest 20 ft. The Arkansas was not so high, but was above medium stage most of the time. There were two great freshets of the tributaries, the former and greater of which occurred before the Mississippi had reached its height. The second took place in April, after the main rise, but during the later rise which succeeded the first, which was nearly as high. The mean of twelve consecutive days at Helena gives a discharge of about 1 470 000 cu. ft. (reduced) at an average stage of 47.24 ft., with a crevasse 19 miles above, discharging about 30 000 cu. ft. The

Arkansas and White Rivers no doubt added some 200 000 cu. ft. to this volume, making in all about 1 700 000 cu. ft. There were many breaks in the levees on both sides of the river.

The flood of 1892 reached a height at Cairo of only 48.3 ft., and remained there only a short time. The year would not have been remarkable had it not been for the tremendous outpour of the Arkansas, accompanied by a very high stage of the White River, shortly before alluded to, which made it extremely formidable, and would indeed have made it disastrous to the levees had it not been for the escape of a large part of the waters of the Arkansas around the head of the Tensas Basin system. The discharge at Columbus is given as about 1 380 000 cu. ft. for a mean of five days, but as the velocities were taken at 0.6 depth, a correction should be applied, which, at Columbus, would amount to 5% or so. At Helena, for six days, the discharge is reported as 1 302 000 cu. ft., but as the meter had not been rated during the season, the results here and at Columbus cannot be relied on. The Arkansas City discharges taken at and after the height of the rise seem to be free from the disturbing influences which had caused so much trouble earlier in the season, and are coherent and consistent with the measurements of the succeeding year. For a mean of ten days at the top of the flood, the discharge is given as about 1 450 000 cu. ft.; corrected for 0.6 depth, by the factor habitually used in 1893 at Arkansas City, about 1 425 000 cu. ft. The overflow around the end of the levee and through breaks along the Arkansas River was estimated at about 280 000 cu. ft. These estimates are always too great. Doubtless the total discharge was between 1 650 000 and 1 700 000 cu. ft.

The flood of 1893 has been pretty thoroughly discussed. The total discharge passing Arkansas City would no doubt have been about 1 600 000 cu. ft.

It is now possible to answer the question propounded on page 475, which was the cause of the discussion of the effect of reservoirs on the discharges of tributaries, namely, whether the increased gauge height produced at Helena by the confinement of the river would be propagated down stream. In 1882, at Arkansas City, the height would have been about 55.8 ft., and the discharge of the two tributaries would have increased the gauge height at Arkansas City about 3 ft. over that of Helena. It was thought probable that the latter would have been, in a confined river, about 53 ft. (see pages 475 and 480). The conclu-

sion is that the increased gauge height at Helena would extend its influence to the other gauges further down stream.

These inferences might be substantiated by reasoning drawn from the relations between the gauges. This part of the subject, however, has already been well and thoroughly discussed by others.

Low-Water Discharge.

Comparatively little attention has been paid to the low-water discharge. Very few observations have been taken at that stage. The least discharge recorded in the reports of the Mississippi River Commission was in the fall of 1891. In November of that year, at a gauge reading of 2.9 ft. at Cairo, the discharge was reported as 114 203 cu. ft. At Elmot Landing, in Plum Point Reach, near the same time, the mean of the four lowest days was about 81 400 cu. ft. At Memphis, at an equally low stage in October, it was reported as about 91 000 cu. ft. At Helena, the lowest record, both in October and in November, for the mean of several days, was about 106 000 cu. ft. at a stage of zero. At Wilson's Point, in November, there was a minimum discharge reported (one observation only) of 117 142 cu. ft. At Red River Landing, the lowest record was 141 179 cu. ft.

The observations of the last four years have shown a perceptible depression of the low-water plane at Wilson's Point, as will be seen by the following table:

Year.	Gauge.	Discharge.
1891	5.00 ft.	133 000 cu ft.
1892	4.7 "	145 000 "
1893	2.1 "	156 000 "
1894	2.3 "	177 000 "

In 1894, this depression was general, and extended at least from Memphis to Vicksburg, the low-water record having been broken at every intermediate point. That this was not due to the small quantity of water flowing in the river is shown by the discharge at Wilson's Point, which was greater than in 1891. What significance is to be attached to this phenomenon is not yet known, and probably further observation will be required to develop it. It may not be permanent. It is to be observed that the stage in 1894 was very nearly paralleled in 1872 at Memphis and at Lake Providence for a day or two. The low water of 1894, however, was far more persistent, the gauge at Vicksburg remaining below zero for nearly three months.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

766.

Vol. XXXIV.—December, 1895.

NOTES ON HIGH MASONRY DAMS.

By JOHN D. VAN BUREN, M. Am. Soc. C. E.

READ SEPTEMBER 18TH, 1895.

WITH DISCUSSION.

No engineering structure requires more scientific care in its design and construction than a high masonry dam. Where the site is a threatening one and lives are at stake, no element of danger against which prudence and skill can provide should be ignored. It is proposed in the following notes to consider some of the dangers and requirements which govern the design of the section of a straight, high masonry dam.

Conditions of Safety.—Five different conditions of safety are to be considered: 1, against overturning; 2, against sliding; 3, against crushing near the toe; 4, against ice and waves; 5, against undermining. After the engineer has fixed upon these conditions of stability or safety, which express his scientific and practical judgment, the design of the section is easily made by the aid of simple mathematical formulas.

Forces.—In addition to the usual forces assumed to act upon the dam, there will be considered a water pressure on the base under full head. This force is usually ignored in designing the section, under the supposition that the masonry can be either made practically water-tight or drained in such a way as to relieve this pressure. It is, nevertheless, always one of the probable dangers to which such a dam is exposed.

Not only does water under a high head percolate with comparative freedom through the best masonry, but shrinkage or settlement cracks or natural fissures give it free paths, to a greater or less extent, into the body of the masonry or foundation—paths always uncertain, both in direction and area. No skill in the construction of large masses of masonry can prevent the existence of such cracks, nor can all dangerous fissures in the foundation be discovered and closed. They may be horizontal or vertical, longitudinal or lateral, limited or extensive. It is, of course, impossible without enormously increasing the section to provide against all dangerous cracks, but by making it heavy enough to withstand a base water pressure, the safety of the structure against such dangers is increased immensely.

In order to provide against or relieve such a pressure without increasing the section, the plan has sometimes been adopted of draining the masonry and foundation. But should the drains be too small for the leakage, the pressure may remain practically unreduced, being reduced only by the head lost in the flow. Again, should the leakage be large, the flow through the drains under a high head may be powerful enough to wear them away, or dislodge the masonry, and thus destroy them and ruin the dam. The violent vibrations caused by the flow of large streams through such conduits under high heads are certainly an element of danger, and such conduits require the very best masonry.

It would seem to be the safer course to omit all such drains, and, by making the masonry and foundation as tight as possible, to keep the leakage divided into small streams, and to make the section large enough to be entirely stable under a full water pressure on the base. The introduction of this force will entirely change the section usually adopted.

That this is not an imaginary or remote danger is proved by the recent disastrous failure of the Bouzey Dam in France, of which the

section was wholly inadequate to furnish the necessary stability, with a water pressure on the base. In 1882, 442 ft. of this dam moved down stream, without tilting, so as to form a bulge concave to the reservoir and having a versed sine of about 1¼ ft., and badly cracking the masonry at the bends. This year, notwithstanding expensive repairs, involving a heavy reinforcement at the toe and a deep cut-off wall at the heel, it was completely destroyed, resulting in the loss of 150 lives. An examination of the section of this dam shows that no provision was made in the design for a water pressure on the base. It is not at all improbable that the equally disastrous failure of the Habra Dam in Algiers, in 1881, resulted from a similar inadequacy in the section.

In the opinion of the author, a water pressure on the base of high masonry dams should always be considered in the design, *i. e.*, the section should be made large enough to provide for a water pressure on the base at all levels.

Foundation or Ground Pressure or Resistance.—The ground pressure on the base is usually assumed to vary in intensity uniformly from heel to toe, being taken as nothing at the heel and twice the average at the toe. Undoubtedly, in large masses of masonry with sloping profiles, this theory is incorrect, and the maximum intensity is nearer the line through the center of gravity. But the theory covers the worst case (and when the masonry may be considered to act as a monolith, it is correct), and it should be followed in the calculations.

The Toe.—Assuming that the maximum base pressure is at the toe, it is evident that the sharp toes provided for high dams in the profiles of Rankine, and in the usual shoe-shaped dams, do not possess sufficient strength ; the masonry at the toe is of comparatively little value for many feet back, as it would crack away from the main body long before the assumed maximum pressure would be reached. The toe should therefore be bold, with a nearly vertical profile.

The Triangular Section.—Nothing practical is gained by considering any other section than the triangle. This is the true shape at all levels for stability against overturning and sliding. The additional masonry required to give proper crest width is too small in amount to materially affect the problem, and the maximum toe pressure can be provided for by altar courses at the base.

A triangular section can be designed so that it will satisfy at once and precisely both the assumed conditions of stability against sliding

and against overturning, and this is readily modified for crest width, ice pressure and toe pressure, without sacrificing economy. It is evident that the most economical section against sliding is a triangle with a toe angle of 90°, and that the best section against overturning is a triangle with an angle of 90° at the heel. Between these two extremes there is a triangle which will satisfy precisely both conditions of stability, which is the required section, subject to modifications for ice and toe pressure.

Design of the Section.

Assuming that the site has been examined and found safe, the problem is to design the section of the dam under the following assumed conditions :

First.—That there shall be a full water pressure on the base at all levels.

Second.—That the ground pressure shall be a uniformly varying force.

Third.—That the coefficient of friction shall be f.

Fourth.—That the pressure at the toe shall not exceed p per square foot.

Fifth.—That there shall be an ice pressure of I per linear foot of dam at the crest.

Formulas.

The Triangle.—Problem : To find a triangular section of a dam in which the base friction with a coefficient of friction f will exactly balance the horizontal forces, and in which the resultant force will pass through the outer end of the middle third of the base.

Let h = height of dam $= 1 = cv$, Fig. 1.

b = base of dam $= r\ h$, r being a coefficient.

$y = v\,b \qquad = r_1 h$, r_1 " "

$x = v\,a \qquad = r_2 h$, r_2 " "

s = specific gravity of the masonry $= \frac{w}{u}$.

*w = weight of a cubic foot of masonry = 156 lbs.

u = " " " water = 62.5 "

f = coefficient of friction = 0.65.

R = resultant ground pressure on base.

*This may be increased a little by including the amount of water absorbed.

The center of moments is taken at P, the center of pressure of the water on the back.

$$p\,b = \frac{b}{3}.$$

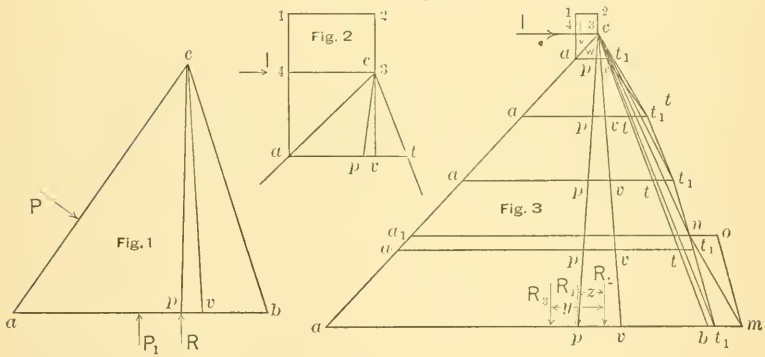
Equations of Equilibrium.

Forces.—Vertical, $\frac{x}{2} + \frac{x+y}{2} \cdot s - (x+y) - R = 0.$

Horizontal, $f\ R = \frac{1}{2}$, or $R = \frac{1}{2f}$.

$$\therefore x + y = b = \frac{1 + f\,y}{f(s-1)} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

$$y = \frac{b f (s-1) - 1}{f} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$



Moments.—

$$\frac{s\,x^2}{6} + \frac{s\,y}{2}\left(\frac{2}{3}x + \frac{y}{3}\right) - (x+y)\left(\frac{x+y}{2} - \frac{x}{3}\right) - \frac{1}{2f}\left(\frac{2}{3}(x+y) - \frac{x}{3}\right) + \frac{1}{6} = 0.$$

Or, as $x + y = b$:

$$s\,b^2 - b^2 - 2b\,y - \frac{b}{f} - \frac{y}{f} + 1 = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

combining (2) and (3):

$$b = -\frac{s-2}{2f(s-1)} + \sqrt{\frac{1+f^2}{f^2(s-1)} + \left(\frac{s-2}{2f(s-1)}\right)^2} \quad \ldots\ldots\ldots\ldots (4)$$

$$y = -\frac{s}{2f} + \sqrt{s - 1 + \frac{s^2}{4f^2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (5)$$

where b and y are expressed as fractions of $h = 1$.

Example.

Let $s = 2.5$ then $y = 0.357\,h$. If $h = 250$: $y = 89\frac{1}{4}$.

$f = 0.65$ $\quad b = 1.263\,h$. $\quad b = 315.8$.

Modification for Ice Pressure, I, Fig. 2, at Crest.—Add a block of masonry, $123a$, at the crest to resist the ice pressure, I, of such dimensions as to give the necessary friction on 34 to resist sliding, and at the same time to pass the resultant of I and the added weight through p, a point in the line of pressure of the simple triangle already determined distant one-third $a\ t$ from t. This block will simply balance the thrust I against sliding and rotation about p. Against sliding from I alone, there will be

$$f\,w\,a\,d = I \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

where a is the crest width of the block 1234; and d its height, 23.

Against rotation about p, there will be

$$\frac{w\,a^2\,d\,(1-2\,q)}{2} + \frac{(w-u)\,a^2\,h\,(2-3\,q)}{3} = I\,h$$

where q is the fraction $\frac{p\ v}{a\ v}$, (Fig. 2).

The triangle $43a$ replaces the same area of water section and therefore adds only the difference of weights corresponding to $w - u$.

Introducing $h = \frac{a}{r_2}$ and $d = \frac{I}{f\,w\,a}$,

it will be found that

$$a = \sqrt{\frac{3\,I\,(2\,f - r_2\,(1-2\,q))}{f\,(2-3\,q)\,(w-u)}} \quad \ldots\ldots\ldots\ldots\ldots (8)$$

Example.

Let $I = 40\,000$ lbs. $r_2 = 1.263 - 0.357 = 0.906$, say 0.91.

$q = 0.07$. $f = 0.65$, say 0.6

$\therefore\ a = 22.3$ ft. and $d = 19.2$ ft.

Areas: $\underline{1234} = 428$ sq. ft.

$\underline{34\,a} = 273$ sq. ft.

Weights: $\underline{1234} = 66\,768$ lbs.

$\underline{34\,a} = \underline{25\,525}$ "

$123\,a \quad = 92\,293$ " $= W$.

The distance of the center of gravity of the added weights, W, from $c = g = 12$ ft.

The values $f = 0.65$ and $s = 2.5$ represent average values. For values of f see Rankine's "Civil Engineering" and the author's paper on "Quay and Other Retaining Walls."*

* See the *Transactions* of the American Society of Civil Engineers, Vol. II, p. 193.

Modification for Ice Pressure below "at," Fig. 2.—Consider h to vary, so that Ih will represent the moment of the ice pressure about p at any level.

Let z = distance which I moves the resultant $R + W = R_1$, toward the toe, Fig. 3. M = moment of W about p at any level. Now let a triangle, ctt_1, be added to the face, with a base $x = tt_1$, in order to equalize the moment Ih, and thus keep p undisturbed.

Then:

$$Ih = R_2 z + M - \frac{r h^2 x}{3}\left(\frac{w}{2} - u\right) \ldots\ldots\ldots\ldots\ldots (9)$$

where R_2 = new resultant $= R_1 + h x\left(\frac{w}{2} - u\right) = R + W +$

$$h x\left(\frac{w}{2} - u\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (10)$$

and $\frac{r h}{3}$ is, for simplicity, taken for the arm of the added weights, which leads to an inappreciable error in the value of z.

In order that the new resultant, R_2, shall pass through p_1, the extremity of the middle third of the new base, b_1, it is necessary that

$$\frac{2}{3} b_1 = \frac{2}{3} b + z$$

or,

$$b_1 - b = \frac{3}{2} z = x, \text{ the required increase.}$$

Introducing these in (9):

$$Ih = \frac{2}{3} x\left(R + W + x h\left(\frac{w}{2} - u\right)\right) + M - \frac{r h^2 x}{3}\left(\frac{w}{2} - u\right)$$

or

$$x\left(4\left(R + W\right) - r h^2\left(w - 2u\right)\right) + 2 x^2 h\left(w - 2u\right) = 6\left(Ih - M\right)$$

$$\therefore x = -\frac{4\left(R + W\right) - r h^2 (w - 2u)}{4\left(w - 2u\right) h} + \sqrt{\frac{3\left(Ih - M\right)}{\left(w - 2u\right) h} + A^2} \ldots\ldots (11)$$

Here A is the first term of the second member.

Now, by the aid of (11), find x for several levels, and join the outer ends of x for a new profile. The computations are set forth in the following tables:

h.	R.	W.	$4(R + W)$.	$r h^2 (w - 2u)$.	$4(w - 2u)h$.	A.
250	3 005 000	92 300	12 390 000	2 447 062	31 000	320.7
187.5	1 715 630	"	7 232 000	1 376 473	23 250	251.8
125	751 250	"	3 374 000	611 765	15 500	178.2
93.75	422 580	"	2 060 000	344 118	11 625	147.9
62.5	187 813	"	1 120 000	152 941	7 750	124.8
31.25	46 953	"	557 000	38 236	3 875	133.9
24	.....	"	.....	.....	.....	.....

$h.$	$I.$	$W.$	$g.$	$3(Ih - M).$	$(w - 2u)h.$	$B.$
250	40 000	92 300	− 4	31 110 000	7 750	4 014
187.5	"	"	0	22 500 000	5 813	3 871
125	"	"	+ 4	13 890 000	3 875	3 584
93.75	"	"	+ 6	9 589 000	2 906	3 300
62.5	"	"	+ 8	5 285 000	1 938	2 727
31.25	"	"	+ 10	981 000	969	1 012

$$x = -A + \sqrt{B + A^2}$$

g = arm of W, from p.

$h = 250 \quad x = 6.2$
$187.5 \quad = 7.6$
$125 \quad = 9.8$
$93.75 \quad = 10.8$
$62.5 \quad = 10.5$
$31.5 \quad = 3.8$
$24 + \quad = 0$

Weight of added area = 260 000 lbs.
Water on base = 100 000 lbs.

Net weight = 160 000 lbs. = W_1

Observe that R varies as h^2, and the other terms as h^2 or h, making the computations simple.

Maximum Toe Pressure.

Let p = maximum allowable (total) toe pressure.
p_1 = " " ground pressure.

or, $p_1 = p - u h_x = 2 \left(\frac{R + W + W_1}{b_x}\right)$.

where h_x and b_x correspond to the level at which p_1 is reached.

Assume $\frac{b_x}{h_x} = r^1$ to be constant.

But $R = \frac{u h^2_x}{2f}$

$$\therefore h_x = \frac{f r^1 p_1}{2u} + \sqrt{\left(\frac{f r^1 p_1}{2u}\right)^2 - \frac{2f(W + W_1)}{u}} \quad \ldots\ldots\ldots\ldots (12)$$

For the simple triangle $r^1 = r$ and $W + W_1 = 0$, giving

$$h_x = \frac{f r p^1}{u} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (12)a$$

$= 189$ ft.

Example.

Let $p_1 = 14\,400$. $r^1 = 1.33$; $W = 92\,300$. $W_1 = 160\,000$.

Then $h_x = 168.5$, corresponding to level $a_1 o$ (Fig. 3).

Below the level $a_1\ o$ corresponding to this height, the base must be increased in order to keep the toe pressure within limits.

Increase of Base Below the Point Where p is Reached.—The resultant pressure is now—

$$R_2 = R_1 + W_1 = R + W + W_1 \ldots\ldots\ldots\ldots (13)$$

when W_1 is the net weight of the portion last added to the front profile.

Now add a block of masonry $n\ o\ m\ t_1$ (Fig. 3) at the toe and extending from the point where the pressure p is reached to the base. Let a provision be made for a reverse water pressure at the toe, having a head h_1, and let the assumption be continued that there is a full water pressure all over the base.

Let R_3 = new resultant = $R + W + W_1$ + weight of toe block $n\ o\ m\ t_1$.

y = distance between R_2 and R_3.

q = distance of center of gravity of block $n\ o\ m\ t_1$ from a vertical through the middle of $m\ t_1$.

d_1 = height of added block.

x = required increase of base.

m = distance of point of application of R_2 from t_1, the new toe = $\frac{b_1}{3}$.

$b_1 = b + 6.4$.

Let M_1 = difference between moment of weight W_1, added on face, and moment of water pressure on the increase of base $b\ t_1 = 6.4$ ft. (Fig. 3).

Then the following three equations of condition result:

$$R_3 = R_1 + W_1 + (w d_1 - u h)\, x$$
$$= R_2 + (w d_1 - u h)\, x$$

$$R_3\, y = w d_1\, x \left(m + \frac{x}{2} - q\right) - u\, h\, x \left(m + \frac{x}{2}\right) - \frac{u h_1^{\,3}}{6} + M_1$$

$$^*p_1 = \frac{2\, R_3}{b_1 + x}\left(2 - \frac{3\,(m + x - y)}{b_1 + x}\right) = \frac{2\, R_3}{(b_1 + x)^2}\,(b_1 - x + 3\,y) \ldots (13)$$

Solving for x

$$x = -\frac{p_1\, b_1 + R_2 - 2 b_1\,(w\, d_1 - u\, h) + 3\, w\, d_1\, q}{p_1 + u\, h - w\, d_1}$$
$$+ \sqrt{\frac{(2\, R_2 - p_1\, b_1)\, b_1 - u\, h_1^{\,3} + 6\, M_1}{p_1 + u\, h - w\, d_1} + A^2} \ldots\ldots\ldots (14)$$

where A = first term of second member.

* Where the formula is the usual one for a resultant acting within the middle third.

Example.

$b_1 = 315.8 + 6.4 = \text{say, } 322 \text{ ft.}$

$m = \frac{b_1}{3} = 107.3.$

$p_1 = 14400.$ $R_2 = 3\,097\,300 + 160\,000 = 3\,257\,300$

$M_1 = 260\,000 \times 60 - 100\,000 \times 104 = 5\,200\,000$

Make $h_1 = 0.$

$$q = \frac{81.5 \times 0.337}{2} = 14.5 \text{ ft.}$$

$\therefore x = 30$ ft.

$y = -3.3$ ft.

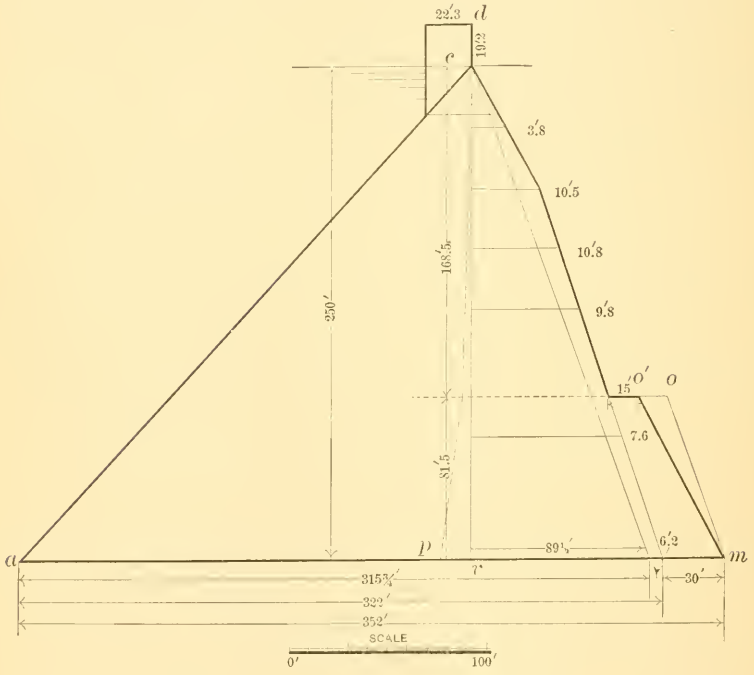


FIG. 4.

This completes the section (see Fig. 4). The block $n\ o\ m\ t_1$ at the toe may, however, without appreciable error, be made triangular, following the line $n\ m$. It will be observed that the inclusion of a back water pressure at the toe will not materially affect the value of x in (14), because its moment for any ordinary depth h_1, which may be taken

as a minimum, will be comparatively very small. But such a pressure will materially increase the stability against sliding below $a_1 n$, and were this alone to be considered, the section might be considerably reduced. By shifting the crest back a small distance on the fixed base $a_1 n$, and thus throwing off some of the overlying water on the back, it is possible to reduce the base slightly at levels above this, and so correct a small part of this excess. But any considerable change of this kind will change the other conditions, and no important improvement can therefore be secured.

The section as now completed satisfies almost exactly the requirements of safety against overturning, ice, toe pressure and sliding. It has only a slight excess of frictional stability between the crest and the base.

Check on the Computations.

The total resultant on the base is—

Simple triangle	= 3 005 000	lbs.
Block 123a at crest	= 92 500	"
Area added to face	= 160 000	"
	3 257 300	"
Less upward effect of toe block $n\,o\,m\,t_1$	= 87 330	"
Total resultant	= 3 169 970	"

Horizontal thrust—

Water on back	= 1 953 125	"
Ice pressure	= 40 000	"
Total thrust	= 1 993 125	"

$$\therefore f = \frac{1\,993\,125}{3\,169\,970} = 0.63$$

But if the toe block, $n\,o\,m\,t_1$, is made triangular, the resultant is reduced 191 710 lbs., half its gross weight, making it 2 979 260 lbs., and

$$f = \frac{1\,993\,125}{2\,979\,260} = 0.66$$

If only half of this triangle is cut off, the toe will have a better shape, the resultant will be 3 074 615 lbs., and

$$f = \frac{1\,993\,125}{3\,074\,615} = 0.65.$$

From (13, second equation) by introducing $x = 30$ there results $y = -3.3$ ft.

$$\therefore p_1 = \frac{2 \times 3\,170\,000}{352}\left(2 - \frac{3\,(107.3 + 30. + 3.3)}{352}\right) = 14\,409 \text{ lbs.},$$

exactly what is required,

The foundation, when not thoroughly good, may be more or less effectively protected against leakage and undermining by an impervious earth filling to any desired height, and such filling, with the section as

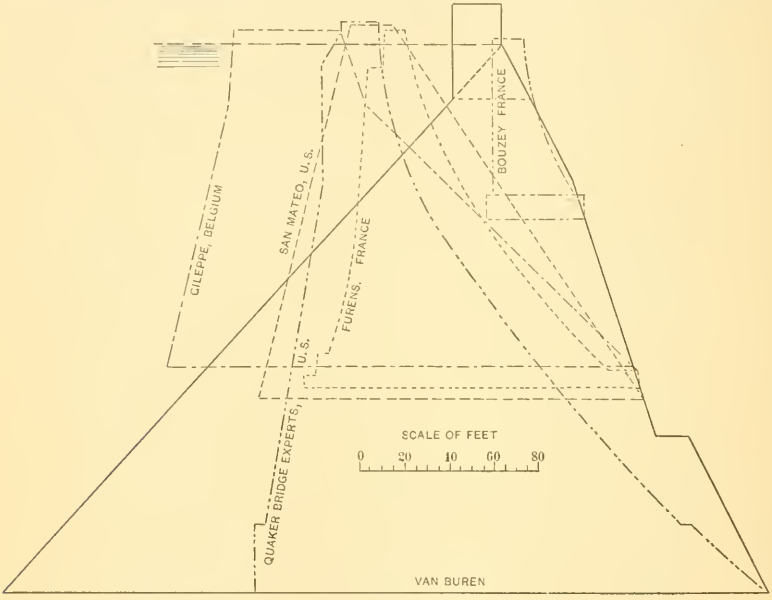


FIG. 5.

designed, will add to the stability in every way. This device, while an additional element of safety, cannot be considered an entirely sure cut-off of water under high pressure. Fig. 5 shows the section drawn to scale, and also the sections of several of the great dams designed in recent years, here and in Europe. Those sections are surely to be commended which, while conforming to theory, provide as far as possible against all dangers. There is no economy without safety, and the engineer who recognizes this truth usually sleeps peacefully on his works.

DISCUSSION.

Mr. Worthen. WILLIAM E. WORTHEN, Past President Am. Soc. C. E.—While the speaker had never built a high masonry dam, he had constructed a number of low ones. In general he had adopted the old form of masonry dam common in New England, which consisted of a mass of common dry rubble masonry, in section a trapezium, with sheet piling on the water side to within 3 to 5 ft. of the crest level, and then sloping up with timber and tight plank covers and often an oak string piece for a crest. But in his dams, with one exception, all wooden constructions are omitted, the sheet piling is replaced by either brick or stone masonry and the cover is of ashlar.

Mr. Lindenthal. GUSTAV LINDENTHAL, M. Am. Soc. C. E.—The speaker believed that there was a simple method of avoiding danger from cracks in masonry dams. It consisted in running iron rods up vertically from firm anchorages in the bed rock to the crest of the dam. They would be 15 or 20 ft. apart, and diminish in size from the bottom to the top, as may be indicated by calculation. These rods would be connected by horizontal ties and the entire iron construction would be bedded in the masonry. If such a dam were constructed, no danger would be caused by the presence of cracks, as the iron network would withstand the tensile strains which might occur. That there was a danger of such cracks arising from other causes than the possible expansion and contraction of the masonry was shown by the earthquake which moved up the Atlantic coast several years ago, being most severe in the South. It injured several reservoir dams and allowed the water to escape from them, resulting in the loss of life and property.

Mr. Ridgway. ROBERT RIDGWAY, Jun. Am. Soc. C. E.—In the construction of the Titicus Dam for the Croton Aqueduct Commission of New York, there was danger of settlement cracks where the core wall passed from a rock to an earth foundation. The chief engineer, Alphonse Fteley, M. Am. Soc. C. E., decided to make the wall weakest there in order to be sure where the crack would occur. Accordingly a well was left in the wall at that point, 5 x 8 ft. in horizontal section, so proportioned that the combined thickness of the two side walls was less than that of the normal section of the core wall. This well was carried up about 100 ft. above the foundation. As expected a slight crack showed itself in the side walls of the shaft, but it was so small that it was found unnecessary to fill the well with concrete, so it was filled with earth and the masonry carried up over it to the crest.

Mr. Skinner. F. W. SKINNER, M. Am. Soc. C. E.—The subject of cracks in masonry is discussed in a recent paper by M. Dutoit on the precautions to be taken in the construction of large reservoirs.*

*See the *Annales des Ponts et Chaussées*, June, 1895.

Mr. Skinner. In this article it is stated that it has been found impossible to build large covered masonry reservoirs near Paris, which would not show cracks caused by the variations of temperature. Consequently the last two Parisian reservoirs, at Montmartre and Villejuif, respectively, have been provided with a double bottom, containing a series of intersecting galleries which allow the condition of the masonry to be kept under constant surveillance, and also collect any water which may pass down through the masonry from the basins above. There is a regular system of patching by which the cracks are stopped as soon as discovered. The plaster that is on top of the concrete bottom is removed for about 2 or 2½ ins. each side of the crack through the masonry, which is thoroughly cleaned out by a steel blade and well washed. A strip of board or cardboard about 0.05 in. is inserted in the crack, and Vassy cement 0.4 in. thick is filled in each side of it. Then the strip is withdrawn, leaving a regular opening, and the fresh cement surface is heated and thoroughly dried by charcoal furnaces, after which 1½-in. strips of pure unvulcanized caoutchouc are spread over the crack on the hot cement and soldered with a solution of rubber in benzine. Then the remaining space, about ¾ in. deep, is filled with cement mortar up to the general surface. This last upper layer of cement usually breaks, but the flow of water is stopped by the caoutchouc, while the open space below it allows for the expansions and contractions of the masonry.

A dam is now in course of erection in California, which has some unusual features that deserve mention, although it is not a masonry structure. It is about 130 ft. high above the bottom of the reservoir and 565 ft. long, and is located in a rocky valley. In building it, a trench about 65 ft. wide was dug 30 ft. deep to solid rock, and filled with good rubble masonry. On top of the masonry and running across the whole width of the valley is a web of thin riveted steel plates from No. 0 to No. 3 Birmingham gauge. As each course of plate was calked and completed, it was coated with a special preparation of asphalt. Burlap was then placed over the top edge and each side of the steel, and a coat of asphalt put over it, making two coats on each side of the plate. On each side of the web built up in this manner, there is a 24-in. concrete wall, and on each face of the concrete is a mass of riprap stone sloped 2½ to 1, giving stability to the dam. Dependence is placed upon the steel web, which is built into narrow slots 4 ft. wide and 4 ft. deep in the rock sides of the valley, for water-tight properties, as the filling of large stone on each side offers little resistance to the passage of water.

The largest rock is placed at the lower end of the slope, and nearly 160 000 cu. yds. of it will be contained in the finished structure.

Mr. Clarke. T. C. Clarke, M. Am. Soc. C. E.—The speaker believed it might be interesting to the Society to consider another kind of dam, which is

simple and inexpensive, and can be used even on poor foundations in cases where the smallest discharge of the river is sufficient to always flow over the crest of the dam and keep it wet. It consists of timber crib-work filled with broken stone and its section is shown in Fig. 6. If the bottom is hard the cribs are sunk directly upon it, but if soft, riprap stone of all sizes is dropped into the river, which settles down through the soft material until it will go no farther. In a sand river this sinking can be aided by hydraulic jets. The riprap should be extended for some distance above and below the dam, to form a protection for the river-bed. Upon this foundation after it has settled, cribs 20 to 30 ft. wide, and as long as the section of the dam requires, should be sunk into place, the greater length of the crib parallel to the current. These cribs are not placed close together, but have spaces 5 to 8 ft. wide through which the summer discharge passes while the superstructure is building. Mr. Clarke.

After these cribs have become fixed in place, the superstructure of the dam is built upon them continuously and with no openings in it,

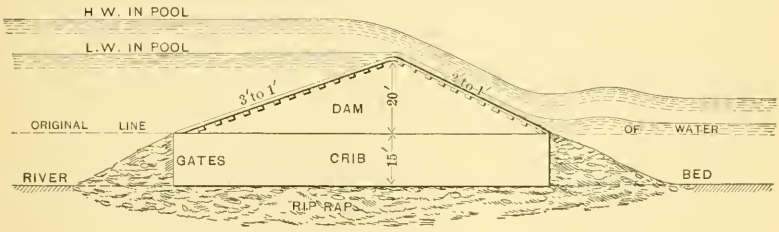


FIG. 6.

clear across the river, and well into the banks. It consists of 12 x 12-in. timbers framed to shape, filled with loose stone and covered on both sides with timbers lying longitudinally up and down stream. They are placed close together and well drift-bolted to the timbers below. The crest of the dam is protected from abrasion by thick plates of rolled iron. After the superstructure is finished, gates of two thicknesses of 12 x 12-in. timber drift-bolted together are placed to close the apertures between the cribs. The water then rises until it flows over the crest of the dam. Such a dam can carry very heavy floods over its crest without injury. Its flat shape on its upper slope makes the water above hold it down, as is the case with dams built by William E. Worthen, Past-President Am. Soc. C. E. The flat slope on the lower side shoots the water up on the top of the water below and prevents its cutting the bed of the river.

A settlement of several feet may take place in some of the cribs without making this kind of dam leak, as it is very flexible. It seemed to the speaker very well adapted for the San Juan River on the proposed Nicaragua Canal, or for such dams as may be required at the foot of

Mr. Clarke. Lake Erie and Lake Ontario to hold up the water to a mean level. A simple and inexpensive sort of material is used that can be had everywhere, and unless the water is allowed to cut around the ends of the dams, floods will not harm them.

Such a dam was built across the Ottawa River by the Department of Public Works of Canada. It is about 1 700 ft. long, and the cribs were sunk in very rapid water from 2 to 23 ft. deep, on a rock bottom. It raises the water 16 ft., and has a lock on one side and a timber slide on the other. It is arched up stream. It has 2½ ft. of water on its crest in summer, and 10 ft. during high floods. No leakage of any magnitude has taken place. Being always submerged, it will last for a very long time.

The speaker designed this dam and many similar ones for the improvement of the navigation of the Ottawa River many years before the structure was actually built. It was very important to know what would be the level of the water at high floods on the crests of the dams, in order to fix the height of the copings of the locks and of the tops of their entrance piers. The following formula was used by him, and also by the engineer of this dam.

Calling the height of water on the crest of the dam H, the length of crest L, the discharge in cubic feet per second Q,

$$\text{then } H = \sqrt[3]{\left(\frac{Q}{3.56\,L}\right)^2}$$

In this case the low-water discharge was 26 000 cu. ft. per second and L was 1 600 ft. Hence $H = \sqrt[3]{\left(\frac{26\,000}{1\,600 \times 3.56}\right)^2} = 2.75$ ft. The actual height observed was 2.50 ft.

The high-water discharge was 190 000 cu. ft. per second, and length of dam 1 760 ft. $H = \sqrt[3]{\left(\frac{190\,000}{1\,760 \times 3.56}\right)^2} = 9.72$ ft. The actual height observed was 10 ft.

This formula is probably as accurate as was the gauging of the river. As this is the test of a very simple weir formula on a very large scale, it seemed to the speaker worthy of record.

Mr. Wegmann. Edward Wegmann, Jr., M. Am. Soc. C. E.—The author of the paper recommends a profile for a masonry dam that differs greatly from all types hitherto proposed. This is caused by assuming an upward pressure under the entire base of the dam, equivalent to the head of the water in the reservoir, a condition not assumed by other writers on the subject. Under the requirements imposed, the section is properly proportioned, the formulas used being very simple. If the profile were designed for an extremely unfavorable foundation, the rock being so fissured and porous as to permit the water in the reservoir to exert

an upward pressure under a large portion of the base of the dam, the speaker had no criticism to offer except the statement that it is not very probable that an engineer would construct a high masonry dam on such a foundation. The author proposed, however, that the section of a masonry dam should always "be made large enough to provide for a water pressure on the base at all levels," a condition which increases the section very much. The speaker could not see the necessity of taking such an extreme supposition for ordinary cases. Mr. Wegmann.

In designing all structures, the engineer has two objects in view, which have opposite tendencies, viz., safety and economy. His skill consists in finding the happy medium which gives sufficient safety with an economical expenditure of money. According to Fig. 5 of the paper, the profile for the proposed Quaker Bridge Dam, designed by a board of experts, would have to be increased about 65% in area to satisfy the author's views. To see whether an upward pressure should always be assumed under the base of a dam, the speaker reviewed briefly what experience had taught about high masonry dams and examined the causes of the failures that have occurred.

So far as he knew, about 55 high masonry dams had been constructed up to the present time. Of this number three had failed, viz., the Puentes, Habra and Bouzey Dams. In each of these cases the failure was due to faulty construction and not to errors in the design. The Puentes Dam was built in Spain during the years 1785–1791. The maximum height of the wall was 164 ft. During the construction of the foundation, a deep pocket of earth was discovered in the center of the valley. Instead of going down to solid rock, the engineers adopted the unfortunate plan of constructing a pile foundation at this place. For 11 years after the reservoir was put into service, the water never attained a greater depth than 84 ft., owing to an insufficiency of rainfall and drainage area. In April, 1802, heavy rains caused the depth of the water to be increased to 154 ft. The result was that the whole pile foundation and soft material in the pocket were forced outward, producing a rupture in the central part of the dam. This accident caused the loss of over 600 lives and the destruction of much property. The failure produced a hole in the central part of the dam, which did not reach, however, up to the crest. The appearance of the central part of the wall after the rupture was that of an arched bridge. Within recent years, this portion of the dam has been repaired and the reservoir has again been put into service.

The second failure mentioned was that of the Habra Dam, constructed in Algiers in 1865–73. A very full account of the destruction of this reservoir wall, with an investigation of the probable causes, has been given by an Italian engineer, Gaetano Crugnola, in the "Ingegneria e Arti di Pareto e Sacheri," Turin, 1882. According to this writer and French engineers who have discussed this case, the failure was not due

Mr. Wegmann. to insufficiency in the area of the profile but to poor material used in the construction and to the strain the wall was subjected to during an unprecedented rain storm. The building material used had to be all obtained near the site of the dam, for reasons of economy. The stone was a very porous sandstone, not uniform in character. The sand used was not very good. The hydraulic lime, made from calcareous stone found on the banks of the Habra River, was not of perfect quality, and contained some quicklime, which probably expanded in the wall and thus assisted in making it porous. Owing to the causes stated, the masonry of the dam was by no means water-tight. When the reservoir was first filled the water passed freely through the wall, which presented the appearance of a great filter. Nevertheless, the dam stood successful for eight years, and did not fail until in December, 1881, when an unusually severe rainstorm occurred, during which 6½ ins. of rain fell in a short time. Crugnola has calculated that in all probability more than 26 000 000 000 galls. of water must have passed over the dam in one night, the sheet of water attaining a height of about 13 ft. above the crest of the dam. This would not have been sufficient to cause the dam to slide on its foundation, but would produce pressures in the masonry near the down-stream face of the wall considerably beyond the limits of safety, without taking into account the strains produced by the impact of the flowing water. The facts stated accounted sufficiently for the failure of the Habra Dam and seemed to the speaker rather to sustain Crugnola's views of the case than the author's statement that the failure was probably due to "inadequacy in the section" adopted for the dam.

The third case of the failure of a reservoir wall was that of the Bouzey Dam, which occurred on April 27th, 1895. An account of the rupture was given in *Le Génie Civil* for May 11th, 1895. The dam was constructed near Epinal, France, and was put into service in 1884. It was founded on porous conglomerate rock, consisting of silicious stones joined together by a very poor cementing material. The foundation trench was carried down to what appeared to be a good bottom, but was not excavated to absolutely solid rock, which was found to lie at a depth of about 20 to 30 ft. in the center of the valley. To prevent leakage under the main dam, a guard wall, 6½ ft. wide, was founded on this compact rock and built up on the up-stream side of the dam. As the maximum depth of the water in the reservoir was to be about 75 ft., this guard wall could hardly be expected to prevent water from percolating under the base of the main dam, causing thus an upward pressure which might destroy the wall's stability against sliding. When the reservoir was being filled for the first time and the water had reached within 11 ft. of the top of the dam, the central portion of the wall slid or bulged forward so as to form almost a regular curve having a chord of about 394 ft., and a versed sine of about 1 ft. There

was no settling nor overturning, but simply a sliding or bulging forward. This deformation of the wall produced numerous fissures in the foundation, which discharged a considerable quantity of water, and caused also four vertical fissures in the front face of the wall. Before the reservoir had been filled, two similar vertical fissures had been produced in the face of the dam by changes in the temperature. Water flowed through these fissures during cold weather, but ceased almost entirely in warm weather, owing to the expansion of the masonry. Some attempts were made to close the fissures by means of tarred, wooden wedges, but without success. In 1888–89 the wall was strengthened by building a kind of flying buttress in front of it, which made sliding in the foundation impossible. The upper portions of the wall were not reinforced, however. In 1890 the water in the reservoir was allowed to rise to its highest level without any change being noticed in the wall. The recent accident occurred very suddenly without any warning. The foundation masonry was not displaced, but a breach about 560 ft. long and about 33 ft. deep below the highest water level, was made in the central portion of the dam. In the center of the valley this breach was cut down to the bottom of the reservoir. The portion of the wall which was ruptured contained four of the vertical fissures that have been mentioned. It seemed astonishing that a dam that had given such signs of weakness had been allowed to stand. Mr. Wegmann.

The failure of the Bouzey Dam is the only case where an upward pressure under the base of the wall is likely to have been the cause of the disaster. In all probability this pressure would not have occurred to any extent if the whole dam had been founded on the solid rock upon which the guard wall was built.

Very few of the dams that have been constructed would be standing if they were subjected to such an upward pressure. As they have remained stable, some of them for many years, it may be safely concluded that they are not subjected to such an upward pressure to any extent. In this connection the Almanza Dam might be mentioned, which has stood for over three centuries.

The speaker could not agree with the author in his statement of the impossibility of making a dam practically water-tight and of preventing cracks and fissures in the masonry. The Aqueduct Commissioners of the city of New York have constructed, within recent years, two masonry dams in the Croton water-sheds that may be taken as fine examples of what can be accomplished by proper care in the construction. The first of these, the Sodom Dam (98 ft. high above the foundation), has been fully described in a paper read before the Society in March, 1893, by Walter McCulloh, M. Am. Soc. C. E.* The second, the Titicus Dam, has a maximum height of 135 ft. above the foundation. In both cases, the front face of the wall appears per-

* See the *Transactions* of the American Society of Civil Engineers, Vol. XXVIII, p. 185.

Mr. Wegmann.

fectly dry in clear weather, there being no signs of leakage. In humid weather some damp spots appear on the walls. Water which may force its way through the pores of the mortar and stone is in a capillary state, and not likely to exert the hydrostatic pressure due to head of water in the reservoir. With the exception of a slight hair crack in the overfall of the Titicus Dam, there is absolutely no crack to be found in the two cases mentioned. The expansion of the masonry is probably taken up by the elasticity of the mortar. As regards settling, the slowness with which a large masonry dam is carried up prevents cracks from this cause.

In the construction of the Sodom and Titicus Dams some fissures had been encountered in the foundation rock, and had been closed by masonry. At these points it was very probable that an upward pressure existed, but the area of such points was a very small percentage of the total area of the base of the wall.

The speaker agreed fully with the author as regards the inadvisability of constructing drains in the foundation. Such channels cause additional leakage, and ought to be avoided. With reference to the danger that might exist if the front face of the dam formed a sharp toe, there was a difference of opinion among engineers. Prof. Rankine stated that the pressures near the front face of a dam were tangent to the curve of the face. If such were the case, the pressures might pass along the front face somewhat as the strains in a masonry arch.

The speaker said, in conclusion, that while the assumption of an upward pressure on the base of a dam diminished the stability of the structure against sliding, the hypothesis being in the direction of safety, it diminished, on the other hand, the calculated pressures in the wall, making the masonry appear less strained than it might actually be. The great danger for a high masonry dam was not a possibility of the wall sliding on its foundation, but the great pressures to which the masonry might be subjected near the faces of the dam. For this reason the speaker recommended that an upward pressure under the base of a dam be never considered in calculating the pressures in the masonry, even if it be assumed, in extremely unfavorable cases, in investigating the stability of the wall against sliding.

CORRESPONDENCE.

Mr. Allen.

KENNETH ALLEN, M. Am. Soc. C. E.—So far as failure from sliding or overturning is concerned, it seems that the author's proposed section would be sufficient for a well built dam of dry masonry. It appears to the writer, however, that the bond and comparative impermeability imparted by the proper use of mortar modifies the conditions radically.

Mr. Allen.

It can hardly be denied that with such masonry as is usually found in well built dams the adhesion of mortar to stone results, to a great extent, in the formation of a monolithic structure, in which the coefficient of friction may generally be disregarded. There are no unbroken horizontal joints of large area, and those which exist are at least nearly full of hard, gritty mortar, clinging more or less to the adjacent stones. Likewise, the assumption of an upward water pressure over the entire bed or base of the dam equal to that of the water at a corresponding depth seems to the writer unjustifiable. Admitting that a certain amount of percolation obtains with an undiminished static head, yet the extreme proportion of such areas of percolation to the entire horizontal section of the dam must be very small in cement masonry. So that, while it is safe to assume a certain amount of upward pressure, as, indeed, will probably obtain, yet this can never act on more than a small fraction of the bed, in the writer's opinion. He is aware that the position held by the author on this point is endorsed by eminent authority, and would not be misunderstood as advocating in any way a section for an important dam based upon any doubtful point without giving the stability of the dam the benefit of the doubt. But as to the influence of cement mortar in increasing the frictional stability and decreasing the total upward pressure, both to an enormous extent, he does not think there is a question. With regard to the calculation of the resultant pressure, he would suggest assuming a certain proportion, say 10, 20 or 30%, of the upward pressure acting on the entire section, depending on the character of the masonry employed, and, in the case of the base, of the bed-rock upon which it is built.

Mr. Duane.

James Duane, M. Am. Soc. C. E.—The area of the cross-section evolved by the author so greatly exceeds that adopted in the best modern practice that it would seem that the premises on which his theory is founded must be in error. This belief is strengthened when it is recalled that in all cases where the materials and workmanship have been good, the generally accepted theory has given ample dimensions. The writer believes that this discrepancy is readily accounted for by the remarkable assumption "that there shall be a full water pressure on the base at all levels." In the few high masonry dams that have come under the writer's notice, all have shown more or less sweating or percolation on the face.

In one or two cases the leakage attained the magnitude of streams at several points, and while this was an eyesore it caused no apprehension as to the safety of the structures. In the worst case where the leakage could be examined at several different points, viz., at the face of the dam, at the interior of a well in the center of the dam, and in the gate-chamber through which the outlet pipes passed, there was no evidence that the upward pressure at any point exceeded a few pounds. This seemed to be independent of the head of water on the particular

Mr. Duane. joint examined, and of the thickness of the intervening masonry between the exposed surface and the water in the reservoir. It may be added that the area acted on by even this slight pressure at any level was insignificant as compared with the area of the entire base at the same level.

Passing now to the generally accepted theory that the base pressure varies in intensity uniformly from heel to toe, being taken as nothing at the heel and twice the average at the toe, the author admits that this assumption is erroneous, and also pleads in extenuation that this erroneous theory covers the worst cases. The writer would ask if the profession generally considers this excuse as a sufficient palliative of the perpetuation of an admitted error. Should not an effort be made to bring this theory into more perfect harmony with the actualities of the case?

The writer believes that while the pressure may be nothing at the heel, that in all normal cases it usually is a definite and considerable amount; that the maximum pressure is at or near the resultant of the various forces acting on the structure; that the pressure at the toe is between that at the heel and the maximum; that the plotted line of pressures would ordinarily be a curve, and that only by the merest accident a straight line, as in the present theory. In dams of insufficient cross-section there may be a negative pressure or tension at the heel, the stability of the structure then being dependent on the excellence of the mortar employed and the time allowed it to set before a strain is put upon it. The writer thinks that it is to the superior quality of the materials employed, and not to any arch action, that such structures as the Bear Valley Dam owe their stability.

Mr. Broomall. C. M. Broomall, Jun. Am. Soc. C. E.—It is to be regretted that the author does not enter into the mathematical treatment of the arch-shaped dam. In view of the severe test to which the Sweetwater Dam was recently subjected, such reference would be of interest. The cross-section of the Sweetwater Dam is very bold, and the extent to which its strength is dependent upon the arch shape must be very considerable. It has 70 ft. of water against it, with 46 ft. bottom width and 12 ft. top width, and is built convex up stream, with a maximum radius of 213.3 ft. Its total length is 396 ft. The fact that for 40 hours floods poured over the top of this dam from end to end is a very strong argument in favor of the use of the curved type. Another strong point in their favor is that on account of the curved form the effects of contraction and expansion, due to changes of temperature, would seem less likely to do damage. Also in the case of cracks caused by unequal settlement, the danger would seem to be much less than in the straight dams. The writer would like to ask what changes the author would make in his formulas in order to apply them to the curved dam. It would appear that the treatment would have to be entirely different, almost reducing to that of an arch loaded normally.

M. S. Parker, M. Am. Soc. C. E.—All engineers must agree with the author "that no element of danger against which prudence and skill can provide should be ignored" in the design and construction of high masonry dams. There is always, however, the question of dollars and cents foremost to capitalists who furnish the means for carrying out the engineer's design. The question arises as to what design is most economical in cost while possessing a sufficient factor of safety, the cost having a considerable influence on the design. The design for a high masonry dam, in the writer's opinion, depends almost entirely upon the character of the foundation to be built upon. Mr. Parker.

There is no disputing the fact that an equilateral triangle is the true shape of a section at all levels for stability against overturning and sliding. This, however, is not the only question the engineer has to decide in the design of such structures. The main question is what design will suffice, in point of safety and economy, to fulfil the prescribed conditions. In addition to the four forces usually considered, the fifth force mentioned by the author, namely, the water pressure on the base under full head, has always been considered by the writer in designs for crib and masonry combination dams for all heights. But in the case of a high masonry dam on suitable foundation, he cannot see the necessity of considering this force such an important factor as to warrant such a change in design as shown between the Quaker Bridge section and the triangular section in Fig. 5 of the paper. It is admitted that water will force its way through the best of masonry, as it will through the best of metal under sufficient pressure, without weakening either material, and that shrinkage or settlement cracks give free paths to a greater or less extent into the body of masonry; but masonry must be very badly constructed that will be so open as to admit water into its interior and hold it under sufficient pressure to endanger its stability to an extent requiring the treatment of the problem proposed in the paper. He is of the opinion that the proper section for a dam should be governed entirely by the character of the foundation to receive it. Admitting the stability of the triangular section for high masonry dams, he questions the necessity of it, assuming the foundation to be solid and suitable to receive the structure.

L. J. Le Conte, M. Am. Soc. C. E.—The uplifting pressure on the base of dams built upon leaky or seamy foundations has been the subject of much study by engineers of late. Much information is being collected bearing upon the questions involved, but it is of a very contradictory character. In extreme cases, such as the author supposes, 250 ft. in height, the additional amount of masonry required to meet the upward pressure on the entire base is something enormous, over 60%, and the necessity of such an increase is a matter of paramount importance. Mr. Le Conte.

Mr. Le Conte. A study of existing dams and the nature of the ground upon which they are constructed will hardly justify an agreement with the author in his broad statements. There are many large earthen dams in California built on leaky ground which have been in constant use for 25 years or more, notably the old Upper Crystal Springs Dam, near San Francisco. The ground at the site was full of water-bearing gravel beds; the puddle trench was carried down 98 ft. below the bed of the valley, and failed to meet a good impervious stratum. Accordingly the proposed height was reduced to 50 ft. above the bed of the valley, and the dam was built. When the lake was filled springs appeared just below the dam, and continued to flow many years. The dam is 30 ft. wide on top and has a back slope of 3½ to 1 and a face slope of 3 to 1; the water is 45 ft. deep.

Likewise the San Leandro earthen dam at Oakland, Cal., now 120 ft. high above the bed of the valley, rests upon leaky ground, and one or more large springs were struck in sinking the puddle trench some 40 ft. or more. This dam, 20 years old, has always leaked more or less around both ends, through the hillsides, as well as up from the bed below.

The dam is 38 ft. wide on top, and has a back slope of 3 to 1 and a face slope of 4 to 1; the water stands 110 ft. deep.

If the amount of upward pressure on the base of these dams was anywhere near that due to a full hydrostatic head in the reservoirs, it is clear that these dams could not possibly exist. The writer fails to see how such a pressure could exist in any case unless there is an impervious barrier located under the down-stream toe of the dam. In natural ground there is every reason to believe that the amount of ground leakage existing at any dam site will certainly continue immediately below the site. Consequently any leak starting at the up-stream toe of the dam will progress down stream with a diminishing pressure to some natural outfall below the dam. Taken as a whole, then, one would suppose that the upward pressure would not much exceed that due to half the depth of water in the reservoir. Again, it does not seem rational to suppose that this pressure is continuously distributed over the entire base, but only at and near the location of fissures in the bed-rock or foundation. The writer certainly believes in piping all the bad springs found in the foundation and taking them to the lower toe of the dam where they may discharge freely into the creek bed.

In designing large masonry dams another matter must be considered, namely, what may be called the middle third fad. In designing dams of moderate height, this law is harmless and near enough for practical purposes, but when it comes to designing a great dam, 250 ft. in height, then the middle third fad becomes a veritable monstrosity. It leads to an enormous waste of masonry and all to no purpose. Just what law to use as a substitute, the writer is not pre-

pared to say, since the whole subject is fraught with speculation as yet; but it is safe to say that if the oblique resultant (empty or full) comes within the middle half of the base, the actual resultant will be, in point of fact, much inside the middle third. All practical and theoretical knowledge points this way. In short the ordinary straight line formula for distribution of pressure on the base of the dam is of no value whatever when it comes to be applied to the designing of large masonry dams. The toe pressure adopted by the author, 7.2 tons per square foot, is too low entirely; good masonry will stand 10 to 12 tons easily. Mr. Le Conte.

H. F. Dunham, M. Am. Soc. C. E.—The author seems to have quite overlooked the advantage derived from the tensile strength of the material used in the construction of masonry dams. It is true that water will percolate through masonry. Imagine a structure like the Quaker Bridge Dam carved out of a single block of porous sandstone; water would find its way through in considerable quantity, but with the water pressure per square inch much less than the tensile strength per square inch of the sandstone, there would be no danger of overthrow. Imagine, now, a horizontal seam dividing the structure at a considerable depth into two parts; the stability would be greatly diminished. But such a seam or crack, not impossible in nature where the scale of operations is proportionately in excess of anything man tries to do, should not be anticipated in any first class masonry construction. It is the function of good work to guard completely against such conditions. It is not evident that settlement or fissure cracks can be of the implied extent. Any one who has had occasion to demolish with drills, bars and dynamite any well built piece of masonry will understand that the "path" area is limited to a very small percentage of the whole area. Mr. Dunham.

A structure like the one referred to might be supposed to rest upon material that had no tensile strength and was quite pervious to water, but no one could fairly be supposed to build upon such a foundation, and it is not usual to generalize for exceptional localities. The foundation and the structure should be properly related, made as nearly as possible like one continuous mass, and the whole treated in each instance so as to insure safety and to avoid needless expense.

John D. Van Buren, M. Am. Soc. C. E.—In defining his position, Mr. Wegmann admits that if the assumptions are correct, the proposed section is properly designed. This leaves only one point at issue, the assumption of a water pressure on the base. On this point the late James B. Francis, Past-President Am. Soc. C. E., in his paper on high dams, says: Mr. Van Buren.

"On the base of a wall founded on a rock, having numerous seams, containing water under a head, the flow from which is not carried off by drains, an upward pressure may be transmitted through the mortar to the entire base."*

*See the *Transactions* of the American Society of Civil Engineers, Vol. XIX, p. 169.

Mr. Van Buren. In discussing Mr. Francis' paper, Joseph P. Frizell, M. Am. Soc. C. E., says:

"No dam, as ordinarily constructed, can be wholly free from action of this kind."*

F. Collingwood, M. Am. Soc. C. E., in the same discussion, says:

"The experiments on transmission of pressures through the mass seem to be conclusive, and certainly point to the necessity of taking account of the lifting force of the water."†

The cases of failure cited prove that water under high head is liable to get under dams built by skilful engineers, making the best use of available material, and with appalling results. It seems like begging the question to contend that the failures were due to poor

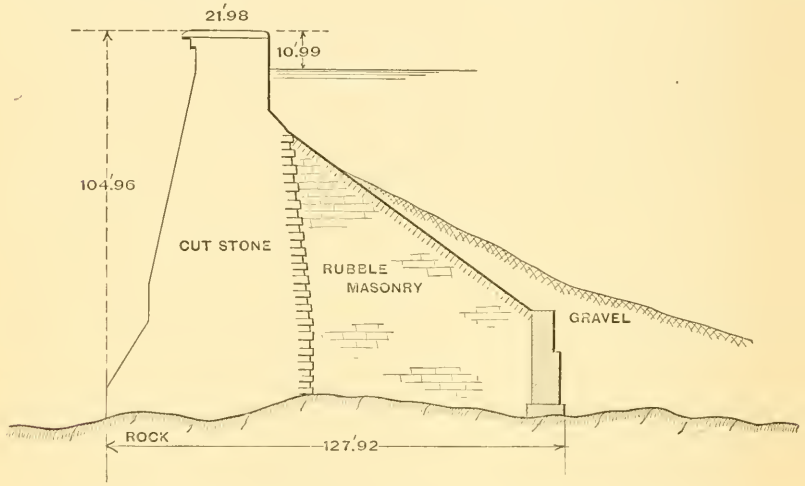


FIG. 7.

material, or that if the rock foundations had been deeper there would have been no failure. Had these dams had such sections as are here advocated, there could have been no such failures.

Mr. Wegmann cites the Almanza Dam, which has stood for three centuries, with a section far below the standard here advocated. But he omits to state that this is a curved dam with a very short radius, 86 ft. How could it be forced out? The (Spanish) Lozoya Dam (Fig. 7), an excellent example of engineering described in Mr. Wegmann's able work on "High Masonry Dams," has a section exceeding the one proposed, which is probably based upon the same theories. It was built by the royal engineers, who evidently appreciated all the dangers which might threaten its safety. Again, the Gileppe Dam has a section greatly exceeding the proposed standard, and, even if it were

* The same, p. 184.

† The same, p. 189.

raised 50 ft., it would still exceed it. M. Bidaut, the distinguished engineer who designed it, recognized all the probable dangers which threaten such a dam, and saw no economy without absolute safety. Mr. Van Buren.

In regard to the probability of dangerous cracks, the remarks of Mr. Skinner are interesting and to the point, also those of Mr. Emery on concrete in the reservoirs of India.* It is certainly impossible to build a large mass of masonry without extensive cracks. In the case of the great Croton Dam, there will be acres upon acres of foundation, and upon it there will be built masonry as high as the pyramids. It requires unlimited confidence to claim that this structure can be built water-tight, or free from dangerous cracks and fissures. Mr. Wegmann cites two recent dams to show that masonry in large masses can be made water-tight. This may be so at first, but the effects of time are sure to develop cracks and leaks. Even absorbed water, under pressure, works its way into streams through the solution and corrosion of the obstructing material. Mr. Allen and Mr. Collingwood have suggested that only a part of the base pressure need be considered, but the difficulty is to know how much should be assumed.

As the author remembers the discussion at the meeting, and as he now reads it, Mr. Wegmann makes an important error in regard to the total toe pressure, confusing it with the ground pressure. The total toe pressure assumed is 30 000 lbs. per square foot, made up of 15 600 lbs. from the hydrostatic pressure, and 14 400 lbs. from the ground pressure. He also makes an important error in stating that the total toe pressure would be increased in case the water failed to penetrate the base of the section as designed. The omission of the base water pressure would, on the contrary, reduce the toe pressure from 30 000 to 25 000 lbs. Mr. Le Conte makes a similar error.

In adopting the theory of the middle third, the author is charged with the "perpetuation of an admitted error." Every theory is, more or less, an "admitted error," and it is necessary to cling to the imperfect theory given by the highest authorities until a better and safer one is advanced. The author knows of none. As to this theory leading to a "veritable monstrosity," it may be remarked that the theory of frictional stability leads to the same result. This is certainly not a fad. The criminal is neither of these, but the assumption of a base water pressure.

It has been suggested that the rôle in designing such a structure should be economy with sufficient safety. In the case of the great Croton Dam, sufficient safety must mean safety as absolute as skill and money can make it. New York cannot afford to take the remotest risk in its construction. The results of a failure would be appalling, the flood wave would sweep to destruction the whole western border of the city, with countless lives, buildings, docks and shipping.

* See the *Transactions* of the American Society of Civil Engineers, Vol. XXXIV, p. 252.

Mr. Van Buren. In conclusion, the author regrets that the old-fashioned type of dam, with the flat slope up stream, has been discarded. As used in masonry, it is in harmony with true principles which forbid tension or bending strains, and utilizes an important element of safety from its shape alone, viz., the weight of the overlying water.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

767.

Vol. XXXIV.—December, 1895.

HOLLOW TILE FLOORS, PAST AND PRESENT.

By FR. VON EMPERGER.

READ AT THE ANNUAL CONVENTION, JUNE, 1895.

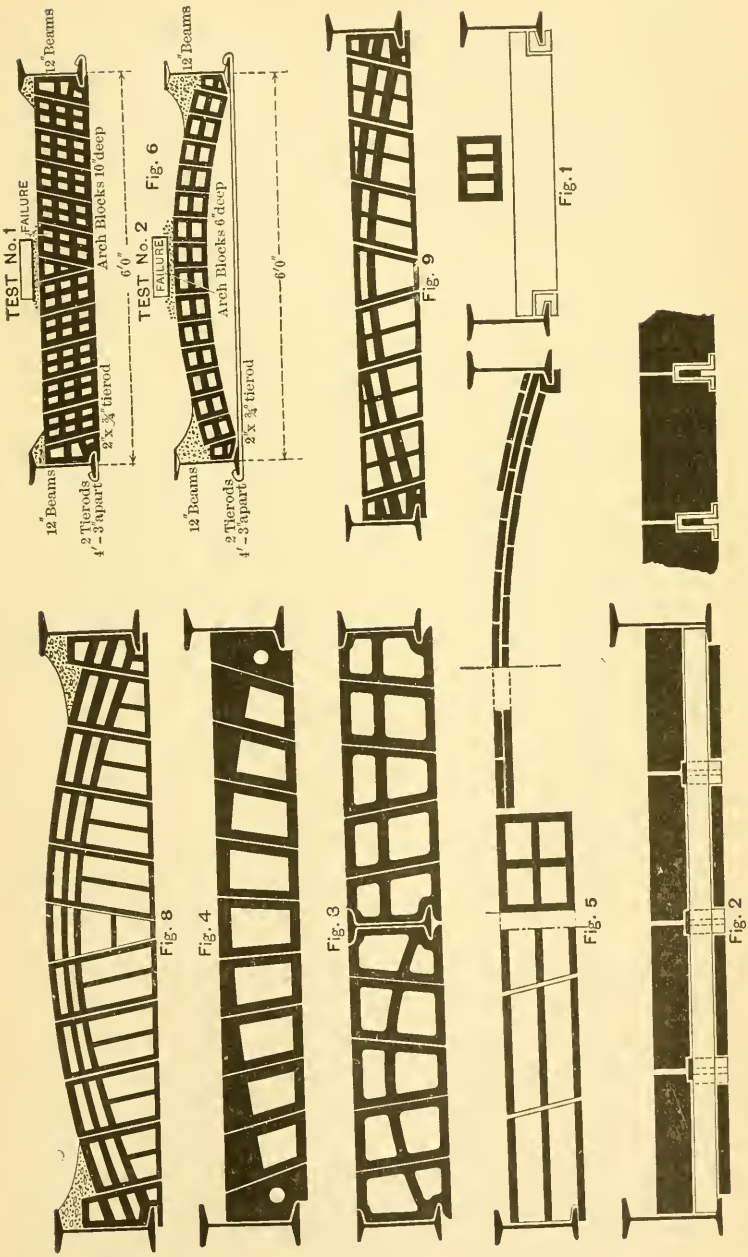
The use of hollow tile for floor construction is scarcely known in Europe. The most recent European designs with flat ceilings have not yet overcome the difficulty of dividing the span into several blocks, and always use one continuous tube of tile covering the whole span, as shown by the French design, Fig. 1. This type is the same as the Liverpool "flags" used in 1853, and introduced again by Homan & Rodgers in 1885. A recently adopted German design shows divisions, but the vertical joints are bound together by imbedded iron rods, as shown in Fig. 2. It is of interest to compare these two recent designs with Fig. 3, illustrating a type proposed in 1873 by Mr. Wear L. Drake, of Chicago, Ill. Notwithstanding the 21 years which have passed, this simple design has not yet found its way across the ocean, and the reason for this is probably the cost of its production, as machine work, which has a most important part in its manufacture, is admittedly cheaper in the United States than in Europe.

The small spans used in Europe, 3 ft. at the most, involve a waste of iron and floor material, and hollow tile could not compete successfully with the concrete construction which has come into such great prominence in Europe as a fire-proofing material, especially as the cement and hand labor used in concrete is comparatively cheaper there.

All European types of hollow tile flat arches could be better termed hollow tile ceilings, as they are designed to furnish a flat fireproof ceiling, concrete being used where strength is needed. The origin of the "old style" hollow tile floor arch is probably indicated by a patent granted to Johnson & Kreischer, March 21st, 1871. The first hollow tile flooring was in the corridors of the New York Post Office and in the Coal and Iron Exchange in New York, built about 1872. Next, it was employed in the Kendall, now Equitable, Building and the Criminal Court Building, both in Chicago. It was used in 1876 in the Chicago Court House, and in 1877–78 in the Van Corlear flats in New York. It seems a well-established fact that its use was not general before the style of building known as skeleton construction was generally adopted, which cannot be assumed to be before 1889. Fig. 4 is taken from an old publication of the Phœnix Iron Company in the early eighties, and it is stated there "as usually made."

Two distinct types of arches have appeared on the market, which have been developed independently and differ in the relative position of the openings and the beams. These openings are either parallel with the beams, as in the "old style" shown in Fig. 6, or are perpendicular to them, as in the "end construction," the latter with its variety of through construction (see Fig. 5), or with separate skewbacks.

Attention is next drawn to the different arrangements of the ribs in the blocks and their development. It can be easily seen that their primary arrangement, as shown in Fig. 4, gives full justice to the engineering aim, that of an arch with a horizontal ceiling below. It is a flat arch with solid blocks, having the superfluous material removed. Mr. Drake had an alternative design (Fig. 3), with horizontal ribs, which are connected in no way with the stresses of the whole construction, and tend only to stiffen the block in its manufacture and handling. The further development has followed the same track with the exception of some well known skewback designs having curved ribs, and some designs and patents which never came into general use.


TEST No. 1
FAILURE
12" Beams
2 Tierods 4'-3" apart
2"x ¾" tierod
Arch Blocks 10" deep
6'0"
12" Beams
Fig. 6
TEST No. 2
FAILURE
12" Beams
2 Tierods 4'-3" apart
2"x ¾" tierod
Arch Blocks 6" deep
6'0"
12" Beams
Fig. 9
Fig. 1
Fig. 8
Fig. 4
Fig. 3
Fig. 5
Fig. 2

Comparing both the Drake arrangements shown in Fig. 3, the author believes, from several independent tests, that with horizontal ribs the section devoted to an arch-like action is comparatively weak, and such a floor will, after some time, carry a load largely by its beam-like action by the help of strong mortar joints, provided they are properly made. This is also proved by the tests with arches 20 days old, made in 1891 by W. E. Cutshaw, M. Am. Soc. C. E., City Engineer of Richmond, Va. In two tests, both with 12-in. blocks, three $\frac{5}{8}$-in. ribs and a span of $4\frac{1}{2}$ ft., one arch cracked in the key at 277 lbs. and broke at 554 lbs. per square foot (arch-like action), while the other showed no crack at all until it suddenly collapsed under 1 000 lbs. per square foot (beam-like action). In two other tests of arches 40 days old, with a span of $3\frac{3}{4}$ ft., made of 12-in. blocks, as above, one broke at 770 lbs., while the other stood 1 005 lbs. per square foot, without cracking.

It is easy for anyone to convince himself concerning these two different actions by closely observing a test of a flat arch. There are two ways in which they may be noticed. In the first cracks, in the bottom of the center do not appear before there is a comparatively high load applied. This will happen when the mortar is strong enough and carefully enough placed to bear the tension stress for a while, which depends upon the setting time and the quality of the cement used and its proportion. The second method of failure will happen with a new, weak or badly made mortar. Cracks appear very soon, going higher quickly and extending up to two-thirds to five-sixths of the height, according to the depth of the part under tension. Only the second case should be considered, following the approved practice never to rely on tension in the mortar joints of an arch and to increase the section devoted to compression according to the load for which it is designed.

It can be seen that in the first case the arch-like action sets in comparatively late, and with a weak arch section the smallest crack is followed quickly by smashing the whole piece; while in the second case with a strong arch section, the results attained are always the same, and cracks may appear at the bottom which are not followed by a sudden collapse.

The arrangement of the ribs shown in the through construction (Fig. 5) is quite a departure, and needs special consideration on account of its simple manufacture. It is the same tube which European manu-

facturers advocate (see Fig. 1). The large span requires an increase of section, and convenience in manufacture necessitates cutting it in pieces with inclined joints. Only the vertical solid ribs transmit strains, the horizontal ribs serving to stiffen the block. With a proper height and width of these vertical ribs any strength desired can be attained; but as the upper part only of these vertical ribs is acting as an arch, it has necessarily to be very strong, and will require about the same amount of material as a solid flat arch.

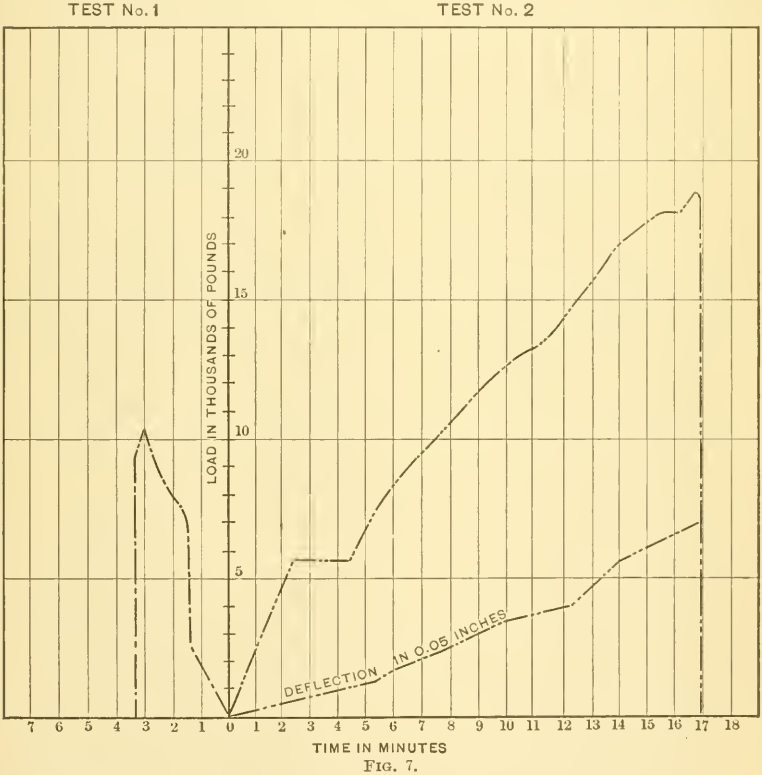


FIG. 7.

The comparative design, Fig. 5, shows that such a hollow tile arch occupies only one-fourth of the section of a solid arch, and if it is to be kept within the same limits, which means to use the same amount of tile, this cannot be done otherwise than at the expense of strength. This ratio, 1 to 4, will necessarily be changed somewhat by the additional material, which stiffens this construction against eccentric loading and gives it a better side bearing than the solid arch, which should

always be tested eccentrically. Efforts have been made to strengthen this style by inserting wires, but this combination of tile and iron did not find favor for several reasons.

The author's efforts were directed to get the desired strength by a different arrangement of the ribs, and to arrange them where really wanted. In the design shown in Fig. 8 an attempt was made to satisfy this requirement by an arch-like arrangement of the ribs by increasing their tile section against the haunches, and thereby stiffening them there. This is, of course, only a development of the older style shown in Fig. 4, and departs from it in details only. The author endeavored to make some tests to substantiate his theory. Inquiries made as to the manufacture of such test arches at the most prominent tile manufactories have shown that they are all willing to have the patterns made if a large enough order is given, and that this type of block with curved ribs (Figs. 8 and 9), could be had as cheap as any other style of hollow tile floor of corresponding section. But for a small experiment only, the cost runs rather high, and the author found a cheaper and as satisfactory a way. Instead of making new blocks, he built the arch shown in test No. 2, Fig. 6, which must at least show the strength of the design as it is given in Fig. 8. The tests Nos. 1 and 2, Fig. 6, were made in connection with others of the Melan type by a hydraulic testing machine, which was built by George Hill, Assoc. M. Am. Soc. C. E., especially for that purpose.

Mr. Hill was asked to select a reliable mason, who was given the design (Fig. 6), with an order to build it in a frame consisting of two 12-in. beams connected by heavy tie-rods. The arches were built January 12th, 1895, which was a mild day, and were tested on January 25th. It may be said that the material was bought in open market from a well-known manufacturer and was put together with the greatest care.

Following is Mr. Hill's report:

"Tests were made using my hydraulic plunger testing machine of 70 000 lbs. capacity; the pressures are registered by means of a Bristol pressure recording gauge which was calibrated before and after the test, and the cards corrected for all error. Deflections were measured with an L-shape multiplying lever multiplying the deflections 10 times and correct within the small range of deflection noted.

"The application of the load was made by leveling up with sand and gravel on top of the arch a space about 1½ ft. by 2 ft., the long dimension being parallel with the skewback beams. On top of this a piece of bluestone 2 x 15 x 25 ins. was placed, and then wooden

blocking. The weight of the blocking, stones and sand was about 200 lbs., and was added to the loads recorded by the gauge.

"All calculations are made with a 10-in. slide rule and may therefore be in error about 2 per cent.

"Test No. 1.—This was a test of a side construction arch of 10-in. blocks, 6-ft. span, 4 ft. 3 ins. wide, parallel with the skewback beams, with two 2 x ¾-in. tie-rods turned over the end of the skewback beams, the section of the arch and its general dimensions being as given in Fig. 6. The arch blocks were of hard tile and represented about a fair average of the usual hard tile blocks furnished to the average building, and were much less strong than the best of hard tile blocks, in my opinion.

"The arch failed without appreciable deflection by crushing in the top horizontal piece or member of the block next to the key, or about 10 ins. away from the center, under a load of 10 600 lbs. The failure was a true crushing of the material and afterwards the first rib down went as well. The loaded area was about 1 sq. ft. The arch area was about 26 sq. ft.; load per square foot, 408 lbs., calculated on total area of arch.

"Compression in the arch can be computed in either of two ways: First, on the assumption that the effective rise is from the junction of the flange of the floor beam forming the skewback with its web up to the center of the second rib down from the top, which may be taken at 5 ins. Second, measuring from the same point to the center of the top web of the arch, in which case the depth could be taken as 8 ins. If we assume 12 ins. as the length in which the compression exists, which is probably less than the truth, we would have a total compression under the first condition of 38 160 lbs., which, distributed on 12 lin. ins., would give 1 410 lbs. per square inch, and a tension in the tie-rod of 12 720 lbs. per square inch. If, on the other hand, we assume that the load was borne entirely by 12 ins. of the top rib, we should then have a total compression of 23 850 lbs. or 2 650 lbs. per square inch. The tension in the tie-rods would be 7 950 lbs. per square inch. From what we know of this material, I am inclined to think that there must have been an initial failure at some one point which carried more than the above amount. If, instead of assuming that the arch failed first immediately under the load, we should assume that it failed entirely across the width of the top member, the fiber strains in the tile would be reduced in the ratio of 4.25 to 1. This assumption I believe to be certainly incorrect from the manner in which the failure occurred.

"The load per lineal foot of skewback, 1 260 lbs., is noticeably higher than that sustained in many former arch tests, and also notable from the fact that failure generally occurs in the skewbacks. The rate of application of the load is graphically shown in Fig. 7.

"Test No. 2.—This was a test of a segmental arch made of poor hard tile blocks 6 ins. deep of section as shown in Fig. 6. Span was 6 ins. Width along skewbacks, 4 ft. Rise, 6 ins. Skews were not well fitted and were not of a very good type. There was some filling on the haunches of the arch.

"The drawing (Fig. 7) shows the rate of application of the load, and also the deflection of the crown of the arch under it, the same spacing representing load in thousands of pounds and the deflections in 0.05 in. The failure occurred just off the center, the block failing along the line shown, one part dropping below the other and indicating a partial shearing action, under a load of 19 000 lbs. No opening at the haunches observed. Calculating the effective rise as 6 ins., the compression on the center of the arch amounted to a total of 57 000 lbs. The tension in the tie-rods amounted to 19 000 lbs. per square inch. The depression in the center amounted to 0.4 in., and the elongation of the tie-rods to about 0.12 in. The load per square foot of arch area was 792 lbs., load per lineal foot of skewback 2 375 lbs., and the effective loaded area about 1½ sq. ft."

By comparing these figures with an equally distributed load as specified by the building laws, it will be found:

That test No. 1 carried, corresponding to the center load.....	816	lbs.	per square foot.
And in addition to that its own dead load of.....	40	"	"
Total breaking load.....	856	"	"
While Test No. 2 carried, corresponding to center load.....	1 584	"	"
And in addition to that its own dead load.....	32	"	"
Total breaking load.....	1 616	"	"

The general practice requires in these structures a factor of safety of ten, but this might be reduced in regard to its own dead load by one-half, a suggestion which is well founded by a corresponding practice in bridge building. Then with a dead load of 75 lbs. per square foot for finished floor (375 lbs.) there remains for Test No. 1 a balance of the breaking load of 481 lbs. per square foot, or a safe live load of 48 lbs. per square foot, and for Test No. 2, 1 241 lbs. per square foot, or a safe live load of 124 lbs. per square foot.

The smallest load the building laws allow is 70 lbs. per square foot in dwellings, so that the floor section in Test No. 1 did not come up even to this moderate requirement, while Test No. 2, having the same tile section, is adapted to bear a load of 124 lbs. per square foot, and is adequate to carry any weight expected in ordinary buildings, especially as the strength would naturally be still increased by an arrangement such as is shown in Fig. 8.

The breaking load of Test No. 1 of 856 lbs. per square foot is about the same as other experiments have shown by careful and repeated testing, although reliable tests are scarce, and most of them are not arranged so as to prove of scientific value.

To illustrate by a single case how even a very faithful experimenter may succeed in getting deceptive results, reference is made to the tests of flooring carried out in Denver in 1890.*

The tests were made by loading the arches with a mass of loose bricks. A uniformly distributed load by such big piles should not be used in these tests, because it represents a solid block which cannot follow the deflection of the floor below it, so that by an unnoticeable deflection the center relieves itself and the load spreads to the haunches. Even if this is not the case, it is never known how the load is really distributed. For this reason, as well as on account of the great convenience in handling, it is believed that Mr. Hill's method will create a turning point to sound practice in testing and consequently in floor construction itself.

The present practice is that an engineer's duty ends after designing the iron-work, and then the architect writes the words "hollow tile floor" across the design. How this hollow tile floor shall be made, how deep the block, and how thick the tile section, does not seem to be considered in connection with the span and the floor load as is done in regard to iron, but is simply a matter decided generally by the cost, sometimes by chance, but always for the convenience of the manufacturer who gets the contract.

The fact that this style of flooring is now used for warehouses and even for floors intended for the heaviest moving loads in manufacturing plants, and that this is done with practically the same 8-in. blocks employed in office buildings indicates that the building laws should prescribe an adequate section according to the load and span, and the

*See the *American Architect*, Vol. XXXI, p. 195.

author submits the following draft of such requirements as a basis for discussion:

	Fig. 9.		Fig. 8.	
Live load. Pounds per square inch	70	100	120	150
Largest permissible span. Inches	5	6	6	6
Size of block. Inches	8	9	10	12
Number of ribs	2	2	3	3
Thickness of ribs at crown. Inches	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{5}{8}$	$\frac{3}{4}$
Thickness of ribs at haunches	Increasing 33 per cent.			
Dead load of tile arches. Pounds per square foot	30	35	40	45

There is already a reduction of dead load in case of small live loads, and it is believed that still smaller dimensions might be warranted by further tests.

It is a remarkable claim for the present style of flooring that by the use of horizontal ribs the arches are not confined to certain spans, but that one and the same block can be so arranged as to fit in any span desired without increasing the costs of setting and manufacture. It is important to state, that by a special arrangement, which would take too much space to explain, the author has succeeded in making the same claim in regard to the curved ribs. There is, of course, no difficulty in giving those blocks a horizontal top, as shown in Fig. 9. In either case there is no need of concrete, except in the haunches in Fig. 8, while it is a well known secret that the concrete cover of the present flat arch is sometimes the only thing which gives it the necessary strength, and that the tile section sometimes used in public buildings is of surprising flimsiness.

There is now a deplorable difference between the tile which the manufacturer uses for his own tests and those bought in open market. There is a further difference in the quality and quantity of the cement used in tests and in regular construction. The mortar used in tests should be of the poorest quality that will be employed in actual work, and it would be the best endorsement of an arch, if, with the use of sand alone, the tests give satisfactory results.

It seems to the author that there is an improvement in the type shown in Figs. 8 and 9, in that the least dimensions are distinctly specified according to the load, and, further, that it is known which

part carries the weight and is essential to strength. It is then easy to see that this part, the arch proper, is composed of faultless material. The remaining part, the suspended ceiling, may have cracks and breaks, which do not matter, and perhaps it could be made either of cheaper material or, at least, of the thinnest tile, to reduce the weight and cost. As no reliance is placed on the joint, the use of a high-grade cement, mixed to give a strong mortar, is unnecessary.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

768.

Vol. XXXIV.—December, 1895.

MOVING TWO 36-IN. WATER MAINS WITHOUT SHUTTING OFF THE WATER.

By E. C. MOORE, Jun. Am. Soc. C. E.

READ OCTOBER 2D, 1895.

WITH DISCUSSION.

The Metropolitan Street Railway Company of New York secured the franchise for a cable road through Lexington Avenue from Twenty-third Street north to the Harlem River, and the author has been the division engineer in charge of its construction. The fact was recognized early that one of the most serious difficulties to contend against was presented by the substructures already in the street. With this fact in view an attempt was made to get an accurate record of exactly what was in the street, by looking up the maps and records of the various gas companies, the city water department and the subway company.

The maps of the gas companies were thoroughly unreliable as to location, as was also the water map; those of the subway company were a little better, but not infallible, so at an early stage in the construction these maps were entirely discarded and no calculations

were made for the moving of obstructions until they were actually encountered in the work. It was found more convenient and less expensive to move obstructions after the excavation had been completed, and the extra expense of a second digging was saved. More obstructions were encountered at intersecting streets than elsewhere, and it was frequently necessary to move two or three outlying pipes or subways in order to get the room required for moving the one which was in the way.

While this system worked very well on Lexington Avenue, where there were no cars to be taken care of on side tracks, it would not answer when the work was extended through Twenty-third Street, where there were not only a large number of cars to be taken care of on the side tracks, but a heavy traffic through the street as well. Moreover, the business houses along the line had to have entrances to handle their wares. Provision was also made for two cables for Twenty-third Street proper, in addition to the Lexington Avenue cables, and the construction problem was therefore a complicated one. All the mains which were parallel to and within the tracks already in use were replaced by new mains, laid on the outside, and in some cases new mains were laid on one side of the street, when there was already a main on the other side, in order to get rid of the house connections, which usually ran too high for the cable construction.

The success of these operations was encouraging and furnished ground for the belief that it would be possible to move two 36-in. water mains in Broadway, between Twenty-second and Twenty-third Streets without shutting off the water, which would not be permitted by the Department of Public Works. These mains were laid in 1839, and had joints every 9 ft.

It was decided that the pipes, which had to be dropped a maximum distance of 2 ft. and moved laterally 1 ft., should be lowered by chains and turn-buckles swung from yokes placed over them at each joint. The yokes were made of 8 x 12-in. yellow pine, the posts being 8 ft. 6 ins. long set on a wooden foot block of 5 x 12-in. yellow pine 2 ft. long, and the cap piece was about 11 ft. long, being 9 ft. between the posts.

The posts were sawed off square, and, after they were set up in place, the cap piece was laid on top of them without framing, then cap and post were fastened together by a cleat of 2 x 9-in. spruce about 2 ft.

long, which was nailed to their sides, one being used at each post and on opposite sides of the bent. Then two diagonal braces about 3 ft. long of 2 x 9-in. spruce were nailed diagonally from the cap piece to the opposite side of the post.

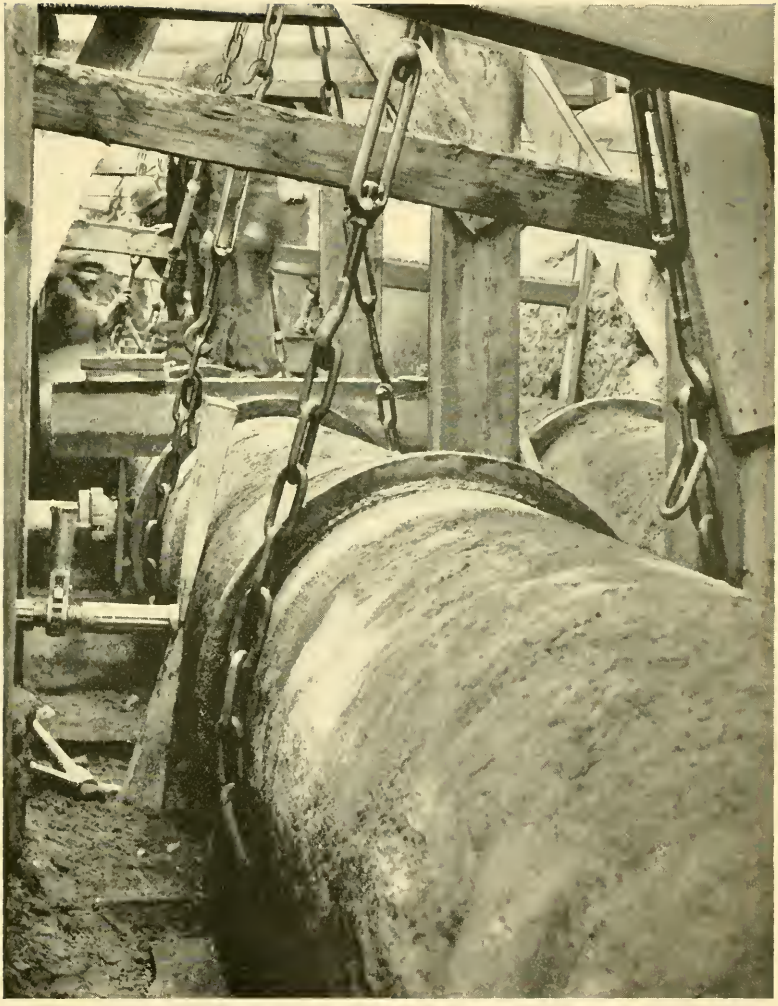
As soon as the first bent was set up and braced from the ground, the next bent was set up, and the two were fastened together by nailing two pieces of 2 x 9-in. spruce on top, reaching from one bent to the other at the ends, and so on until twenty bents were in place.

Then the chains, which were of $\frac{3}{8}$-in. round iron with links 7 ins. long on the inside, were put in place (see Plate VII). The chain which went over the yoke was 5 ft. long, and that which went around the pipe was 11 ft. long. There were two turn-buckles at each bent, so that the chain would not have to slip around the pipe or the timber in lowering. Each turn-buckle was of $1\frac{1}{2}$-in. iron, with hooks on each end, and could lower 12 ins. without changing hooks. The hooks were made to just fit in the links of the chain, and when the pipe was dropped as far as the turn-buckle would go, each turn-buckle was taken off in turn and the hooks placed in links nearer the end of the chain, when the lowering proceeded as before. Everything was made of sufficient strength to allow of every other turn-buckle being taken entirely off without blocking the pipe. This proved of great advantage, as much time would have been lost if it had been necessary to lower the pipe down on blocks in order to change the turn-buckle.

When the lowering of the pipe began, it was decided to move it sidewise at the same time. This was accomplished by hanging the chains over the cap piece a few inches to one side of a perpendicular through the center of the pipe, and putting a brace of 4 x 6-in. spruce from the end of the cap pieces to the bank, to prevent the carrying over of the top of the bent toward the pipe, instead of the desired movement of the pipe in the opposite direction. At the same time, a brace of 2 x 9-in. spruce was added at each end of each bent, running diagonally down from the top of one bent to the bottom of the next, and in opposite directions at the opposite ends of the bents. This was to take care of the torsional strain caused by the cap piece being at right angles to the pipe. By moving the chains along the cap piece a few inches at a time, it was found that when the pipe was down it was exactly in position sidewise also.

By beginning at one end, it was possible to lower the first 100 ft.

PLATE VII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXIV. No. 768.
MOORE ON MOVING TWO 36-IN. PIPES.



of the pipe into position, and to take down and move ahead the first eleven bents; then, as the pipe came into position, each bent was taken down and moved ahead again. The two pipes lay parallel to each other and 5 ft. apart from center to center. One pipe was lowered at a time. There was one hydrant connection on each pipe; this was cut off outside its gate, and a clamp of wood was fitted around the pipe and a long arm attached which reached out to the cut in the connection, and gave it support at that point. Clamps were put around the joints in the hydrant connections, to prevent any possibility of their blowing out.

It was found that almost no calking was necessary during the lowering of the pipes; there were occasional small leaks, but they generally stopped themselves as the pipe was lowered and the joint had a chance to come back into place. It was thought desirable to do no more calking on the pipe than was absolutely necessary until the work of lowering was entirely finished; then all the joints were gone over and put in good condition.

As it was desired to push the work as fast as possible, a connection was made with an electric lighting cable, and eight arc lights and about twenty incandescent lights were set up, and the work proceeded night and day. Excavation began on June 19th, and the work of putting in the bents over the mains began on June 22d. Both pipes had been moved and were in place June 29th, and within another week the trench was filled and paved over, except where the line of the excavation for the cable tracks crossed.

The pipes were moved the extreme distance for 100 ft., and 100 ft. was allowed at each end of this distance for the gradual swing, so as not to strain the joints too much by short bends. This made a total length moved of about 300 feet for each pipe.

The approximate cost of the moving was about as follows:

40 1½-in. turn-buckles with hooks on each end..	$102 40
320 lin. ft. ⅞-in. B. B. chain, 7-in. links.....	102 95
Lumber, including bridging and wedges.....	298 74
Electric lights.....	175 00
City inspectors.....	463 62
Engineers.....	109 00
Labor, carts, etc.....	4 250 00
Total.....	$5 501 71

The foregoing cost may be considered somewhat high, but while the main object sought was the completion of the work in the shortest possible time, every effort was made to avoid unnecessary expense. Night work adds very materially to the cost of such undertakings, because a man cannot do as much work at night as in the day time, and there is also more opportunity to shirk. The cost of providing lights and other conveniences and safeguards must be considered.

DISCUSSION.

Mr. Wegmann. EDWARD WEGMANN, Jr., M. Am. Soc. C. E.—The speaker stated that there were other cases than those mentioned in the paper in which 36-in. mains in New York City had to be shifted and lowered to a considerable extent.

When the old Croton Aqueduct was constructed, the masonry conduit terminated at a receiving reservoir located in what is now Central Park. From here two lines of 36-in. mains, laid in Eightieth Street to Fifth Avenue, and then down this avenue, conveyed the water to a distributing reservoir at Forty-second Street. After the mains had been laid, changes were made in the grades of Fifth Avenue, which necessitated the lowering of the pipes at some points. The first case of this kind occurred in 1850, at Murray Hill, near the distributing reservoir. At this place the pipes had to be lowered 16 ft. The Commissioners of the Croton Aqueduct Department decided at first to remove the mains until the avenue had been excavated to the new grade, and then to relay them. If this had been done, the only connection between the receiving and distributing reservoirs, for the time being, would have been a line of 30-in. mains that had been laid the preceding year in Third Avenue. Mr. E. W. Tracy, one of the engineers of the Aqueduct Department, who subsequently became chief engineer of the Croton Aqueduct, proposed a practical plan for lowering the pipes in place, which was accepted and successfully carried out under his direction.

At the place in question the material to be excavated was hard pan. Small drifts, located about 8 ft. apart, were excavated under the mains. In these drifts blocking was placed to support the weight of the pipes. After this had been accomplished, the material remaining between the drifts was excavated. By using jack screws, the blocking was then gradually removed and the pipes lowered to the new grades, the additional length being obtained by the pipes drawing a trifle out of the joints. Only one line of mains, from which the water was shut off, was operated on at a time, the other line being kept in service.

Mr. Wegmann.

Another case of this kind occurred in 1856, on Fifth Avenue, between Forty-seventh and Fifty-fourth Streets. In this instance, the two lines of 36-in. mains were lowered a maximum distance of 11 ft. without the water being shut off from either line. As the excavation had to be made in solid rock, the pipes had first to be shifted to one side. This was accomplished after the avenue on the east side of the mains had been excavated to the new grades, by placing under the pipes wooden saddles that rested on greased skids. The mains for a length of 1 300 ft. were then moved over to the east, by means of jack screws, a maximum distance of 10 ft. After a pipe trench had been excavated in the middle of the avenue, the pipes were moved back to blocking which had been placed in this trench. The lowering of the mains was then accomplished in the manner explained in the first case. During the shifting and lowering of the pipes but very little calking of the joints was required. The work described, which attracted considerable attention at the time, was performed under the direction of the late James P. Kirkwood, Past-President Am. Soc. C. E., subsequently chief engineer of the Brooklyn Water-Works.

In 1870, the two lines of Fifth Avenue mains had to be lowered between Sixty-eighth and Seventy-second Streets a maximum distance of 4 ft. The work was performed successfully under the direction of Gen. George S. Greene, Past-President Am. Soc. C. E.

Mr Owen.

James Owen, M. Am. Soc. C. E.—While the speaker had no experience in moving large water mains, he had lowered many small pipes without shutting off the water. The first he undertook to move was an 8-in. main that was to be dropped a number of feet. Elaborate preparations were made for doing the work slowly and uniformly, but at the last moment something gave way and part of the line of pipe fell to the bottom of the trench. It was with considerable anxiety that he descended to examine the condition of the main, but when it was found to be intact and tight, he experienced a change of feeling and has ever since felt confident of success in such undertakings.

Mr. Platt.

Joseph C. Platt, M. Am. Soc. C. E.—In the summer of 1894 one of the 36-in. pipes of the Brooklyn Water-Works was uncovered, and a 12-in. hole cut on one side in order to connect a pipe of that size with it. This 36-in. pipe was one of the original mains of the system. It was made in Scotland and laid as early as 1859, and was probably well covered with the genuine Dr. Angus Smith's coating.

The piece which was removed was free from tubercules, pitted holes and all other evidence of deterioration.* In fact, it appeared as good as many lengths which have lain in a pipe yard for a year. The iron was strong and clean, a good pipe or machinery iron, and quite different from much that is of later date.

* This piece is now (October, 1895) on exhibition at the office of the Water Purveyor of Brooklyn.

Mr. Platt. It has been stated by those in charge of the Ridgewood station of the same water-works, where the water is pumped before it runs through these 36-in. pipes, that the pump cylinders and chambers handling this water are so deteriorated that the iron will not hold a tap bolt, but that it is necessary to use bolts running through the metal and a packed washer on the inside, whenever it is desired to make an attachment to the parts in question. The speaker did not attempt to decide whether the great difference in the lasting qualities of the two irons was due to the fact that the pipes were coated and the machinery was not, or to a difference in the metal originally, or to the frequent vibrations of the pumps, from which the pipes are free, or some other cause.

CORRESPONDENCE.

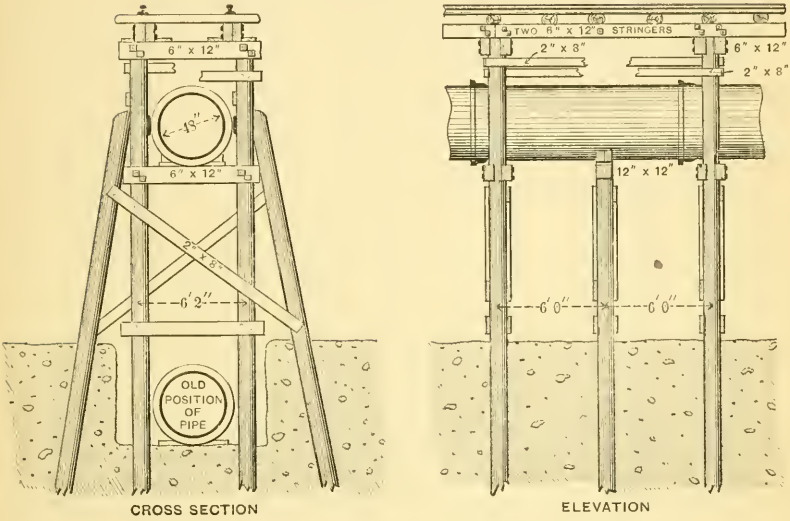
Mr. Brackett. DEXTER BRACKETT, M. Am. Soc. C. E.—The writer has had occasion to move a number of large water mains, and as the methods adopted have been somewhat different from those described by the author of the paper, a description of the work may have some value to others in similar undertakings.

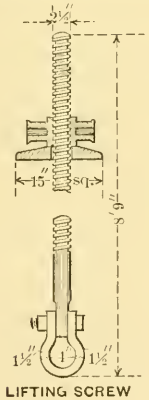
In 1885–86 the grade crossings of Brookline Avenue and Beacon Street with the Boston and Albany Railroad were abolished, and the streets carried over the railroad. Each of these streets contained one of the large mains supplying the city of Boston, which, if not raised, would have been buried 20 ft. below the surface near the railroad, the grade running down and meeting the old grade of the street about 900 ft. from the bridge in either direction. Both pipes were raised in substantially the same manner, so that a description of the method of raising the 48-in. pipe in Beacon Street will suffice for both.

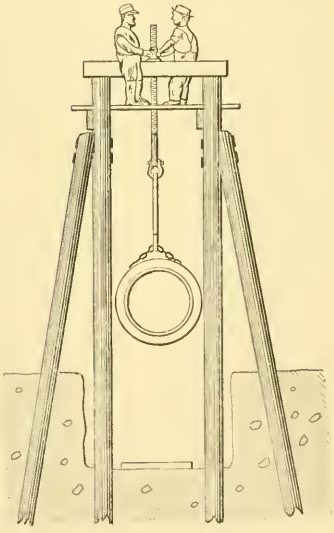
The bridge was first built, the pipe being left in position and the masonry of the bridge abutments built around it. Piles were then driven at intervals of 6 ft. on each side of the pipe for the entire distance to be raised. Alternate bents of the piles were capped 6 ins. below the new grade of the street, with 6 x 12-in. double girder caps. The intermediate bents of piles were cut off about 12 ins. below the new grade at the bottom of the pipe, and fitted with 12 x 12-in. caps, which, however, were not placed in position until after the pipe had been raised.

The earth was then excavated from around the pipes, which were raised by means of screws 8 ft. 9 ins. long and 2.5 ins. in diameter, placed 12 ft. apart. These screws passed between the 6 x 12-in. girder caps, and were supported by a cast-iron plate 15 ins. square, which rested on the caps. The distance between the chains, which were placed around the pipes, to the bottom of the screws, was made up

Mr. Brackett.







CROSS SECTION SHOWING METHOD OF RAISING PIPE

FIG. 1.

Mr. Brackett. where necessary by long hooks made of 1-in. steel. The dimensions of the pile bents and lifting screws are shown in Fig. 1.

When everything was in readiness, the pipe was cut at the bridge abutment, and the entire length of 900 ft. to the foot of the grade was raised in about four hours. The 12 x 12-in. caps were then placed in position and the pipe joints redriven. The piles which had been used to support the lifting screws were girder-capped under the pipe, but the wedges between the caps and the pipe were not driven until the street had been filled to the grade at the bottom of the pipe, for the reason that the filling was done from cars run on the trestle, and the jar caused by running and dumping gravel trains would have been likely to cause leaks in the pipe. But one-half of the 1 800 ft. could be raised at a time on account of the number of lifting screws available. These screws have been in use by the water department for many years, and have proved of great value on many occasions.

In 1889 about 1 000 ft. of 48-in. pipe was lowered, and in this case a much simpler and cheaper plan could be adopted. The pipe was lowered a maximum distance of about 2 ft. in the following manner: The trench was first excavated to the depth of the bottom of the pipe. Cross trenches were then dug every 12 ft., in each of which was placed a blocking, the top of which was at the new grade of the bottom of the pipe. Laborers were then placed on each side of the pipe, and the earth gradually removed from beneath the pipe, thus lowering it to the required grade. The time occupied in lowering 1 000 ft. of pipe and redriving the joints was but about eight hours. Although the gates on the line were closed, the water was not drawn from the pipe, and the writer would have had no hesitation in maintaining the flow through the main if it were necessary. In this case no screws or blocking were used.

On another occasion a 40-in. main crossing a bridge was lowered while in use, but in this case the pipe was supported and lowered by screws.

Mr. Moore. E. C. Moore, Jun. Am. Soc. C. E.—While the moving of large water mains, both with and without the water on, and sometimes the pressure, is no novelty, yet each particular case presents its own problem, and it is quite possible that the most careful canvass of past methods may not furnish an exact parallel. A certain amount of originality therefore becomes a necessity.

This was especially the case with the work described by the author. In the first place, the number of lineal feet of pipe to be moved was comparatively very small. The cost of excavation and preparation for moving was very large, due to the fact that the work was located in one of the busiest points along Broadway. The matter of public convenience had to be given serious consideration, and, therefore, it was necessary that the swing of over 2 ft. be made in the shortest distance

in which it could be safely done. After careful calculations, it was decided that eleven joints on either side of the part moved would furnish the necessary deflection without dangerously pulling any of them. The conclusion proved to be a correct one, for some of the joints were certainly pulled all that they would safely stand, and yet there was no accident to any of them. Mr. Moore.

As to the method employed, there were several reasons for its adoption In the first place, some of the possible consequences of a break in the pipes had to be considered, the most serious of which would be a fire in the downtown district, in fact almost anywhere below Fourteenth Street, in the proximity of Broadway, in which district one of the mains moved furnishes almost, if not quite, the sole supply. It would have been hardly possible to repair any break and get the water turned on in less than four hours, even if everything went smoothly and every one of the old gates worked perfectly, which is hardly probable. Any one of the dozen which would have to be shut and opened again failing to work might make the four hours twenty-four. There is no use in wasting figures on the possible amount of damage a conflagration might do in the meantime. Going from the possible to the certain, a break of any appreciable extent would have undermined the Broadway cable tracks, flooded all the adjacent cellars and blocked all traffic in Twenty-third Street and Broadway. Therefore it was considered necessary by the author to take every precaution possible and spare no expense which might add an extra safeguard, and it is with all these facts in view that the author considers the successful carrying out of this work worthy of record.

In the discussion there seems to be an idea prevalent that all past attempts of this kind have been successful and therefore the actual danger was very small, but such is not the case. While the author has not looked up the records of work of this kind he knows from his own experience that it is always dangerous to attempt to do anything with old pipes, for age seems to make them very hard and very brittle. In Lexington Avenue the 6-in. water pipe, which had to be moved all along the line, frequently broke and sometimes would not admit of any movement whatever. Mr. G. W. Birdsall had the same experience in moving old mains in the city, it frequently being the case that a very small movement would split the hubs. Therefore the author considers it certainly unsafe to attempt the moving of a main with the pressure on, without first being sure that the pipe is in a condition to stand the strain.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

769.

Vol. XXXIV.—December, 1895.

TESTS OF FIRE-PROOF FLOORING MATERIAL.*

By GEORGE HILL, Assoc. M. Am. Soc. C. E.

READ DECEMBER 4TH, 1895.

The discussion of Mr. Fr. von Emperger's paper on "The Development and Recent Improvement of Concrete-Iron Highway Bridges" † brought out very clearly that great numbers of tests of the materials entering into building construction have been made, and the characteristics of the materials very accurately determined; yet the subject of floors had received relatively but little attention, and that not of a kind to indicate with any certainty how strong any particular form might be, what the principles were which governed the construction of a floor, or how the materials in use in this country could be so shaped as to develop the maximum of strength with a minimum of

* The Discussion and Correspondence on this paper will be printed in a subsequent number.

† See the *Transactions* of the American Society of Civil Engineers, Vol. XXXI, p. 457.

material. These facts were already known to the author from investigations which he had made in order to determine with some degree of accuracy what the factor of safety of floors was, and what the most advantageous disposition of the material might be, taking the shapes which were on the market. He therefore determined that, after reviewing the subject generally, bringing together as many published tests as possible, he would make a new series of tests from the engineer's point of view, to determine something positive regarding the subject. He had in mind at the time principally those forms of floor construction which furnished a flat platform to work on above, a flat ceiling below, and had for their principal constituent clay in some form or other, and therefore consulted with about a dozen manufacturers of fire-proof floor arches. These at first demurred and were very indifferent, then became interested, and finally all promised hearty cooperation, and that they would build nine or more arches according to certain rules which the author would lay down and under the supervision of his assistants, which arches should be tested to destruction, it being understood that their only expense would be for building the arches and furnishing men to handle the testing machine when breaking them. Mr. von Emperger also desired to have the Melan system represented in such a test, and, consequently, instructions were prepared in conjunction with him for the building of a series of Melan arches.

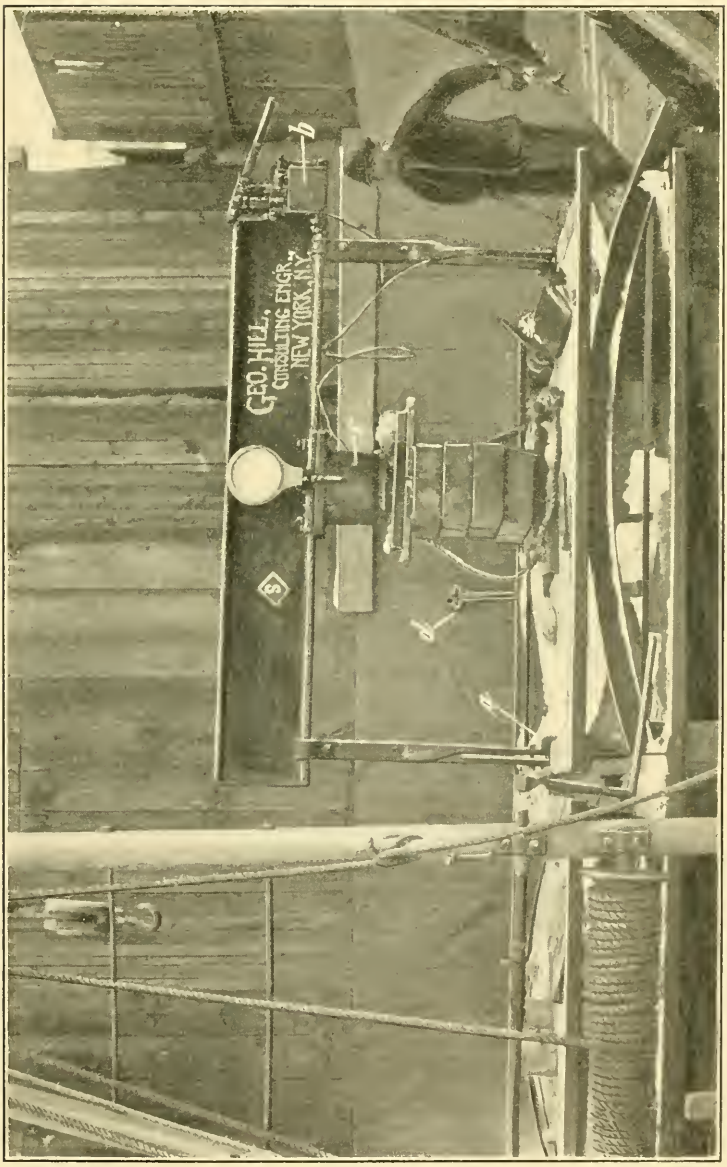
After settling these preliminaries satisfactorily, the author designed a hydraulic transverse testing machine, which was built for him by the Watson & Stillman Hydraulic Works. Considerable time elapsed while these matters were under discussion and the machine being built. When the machine was approaching completion, several of the manufacturers who had promised to build arches found that their interest had diminished, or their cash was threatened by the prevailing "hard times," or something else had happened, to such an extent that they neglected to build the arches altogether. The Melan system arches were, however, built during October, 1894. The Raritan Porous and Hollow Brick Company built several series for test in 1895, and quite a number of arches were furnished by the Staten Island Porous Terra Cotta Lumber Company for test in 1894 and 1895, and it is hoped that certain other manufacturers will build arches and have them tested prior to the discussion of the paper, at which time data

will be given concerning any additional tests which may have been made.

Testing Machine.—A very thorough study of the tests which have been made heretofore of fire-proof floors shows that one of three forms of loading has been generally adopted: 1, pig iron; 2, piles of brick; 3, hydraulic pressure as used by J. B. Johnson and Edward Flad, Members Am. Soc. C. E., in some tests which they made in St. Louis.

In the comments in every case following these tests where brick or pig iron was used, mention was made of the difficulty encountered in handling the loads where the arches possessed high resistance. In the arrangement used by Messrs. Johnson and Flad, it is self-evident that unless water at high pressure can be obtained, the apparatus is of limited utility, and it is further evident that it would be applicable to a very small range of spans. There was also noted in all cases a difficulty in observing satisfactorily the way in which they acted immediately before failure, and the author therefore determined to design an apparatus that would be of general use for testing floors in buildings over any of the usual spans, would be portable, would itself write the record of the application of the load, and would apply the load in such a way as to make it feasible to examine minutely the arch under test.

The machine was designed and built in the following manner (see Plate VIII): A 15-in. I beam, 80 lbs. per foot, 7 ft. 9 ins. long, was used as the principal member. It was supported above the I beams, which form the skewbacks of the arches by means of two pairs of wrought-iron clamps, shown at *a* in Plate VIII. On one end of the large beam a small hydraulic pump *b* was placed, and connected to the pump by a long coil of copper pipe was a ram *c* for putting pressure on the arch. The pump was $\frac{3}{4}$ in. in diameter of piston and $2\frac{3}{4}$ ins. stroke, and one man could get a pressure of 1 500 lbs. per square inch, or 75 000 lbs. on the arch, quite easily. It was one of the standard Watson & Stillman pumps with brass valves, and never gave the slightest trouble. Once in the middle of the test some sticks became mingled with the oil which was used instead of water in order to prevent freezing, and one of these small sticks lodged under the valve; the valve was taken out, cleaned and replaced without any difficulty whatever. The ram was of 7-in. stroke, about 8 ins. in diameter and exactly 50 sq. ins. in


GEO. HILL,
CONSULTING ENGR.
NEW YORK, N.Y.

area, and had the bottom crowned about $\frac{5}{8}$ in. It was set in a cast-iron body and was packed with leather. It was arranged to hang on the under flange of the 15-in. I beam by means of a couple of clamps, which could be tightened on it, and could be slid along to any desired position on the beam. To the upper part of the chamber of the plunger a set of three valves was connected, to which three Bristol pressure-recording gauges were attached. One had a range of 3 000 lbs., one of 1 500 lbs., and one of 400 lbs. per square inch. The readings were, therefore, to be multiplied by the area of the piston, or 50, to get the actual pressure exerted. The dials were rotated by clocks, making one revolution per hour, and so both the amount and the rate of the application of the load were recorded graphically. Two gauges were used in conjunction at all times, thus checking one another. In the tests made they always agreed closely.

In plotting the tests, the curved co-ordinates were made rectilinear and enlarged about 30%, so as to facilitate the reading of the results. The clamps were first made of good wrought iron, bent to shape, but under a pressure of 50 000 lbs. these clamps broke at a point marked *d* in Plate VIII, the metal going clean off, but with a good and rather fibrous fracture. They were then increased to 3 x $1\frac{1}{4}$ ins., and under a pressure of about 70 000 lbs., eccentrically applied, as will be seen in the loading of Test No. 87, they snapped again. They were finally made $\frac{3}{4}$ x 6 ins. at the clamp and $\frac{3}{4}$ x 3 ins. higher up, and these have sustained a load of 125 000 lbs. without signs of strain. With the exception of these clamps, the machine worked so adequately that no changes of any kind suggest themselves as desirable. For handling this machine either a stone-setter's derrick or a small boom derrick or a chain block carried by a stand similar to the four-legged stand used by the weighers in the street and mounted on small wheels may be used, and a small screw jack is needed to push back the plunger after it has been extended its entire length. With such an arrangement a recorder and two men can readily break a series of twelve arches in a day, making all necessary observations.

Melan System Arches.—Mr. Fr. von Emperger, representing the Melan system in this country, had a series of arches built in accordance with the system in the author's presence, between October 10th and 27th, 1894, for the special purpose of having them tested to destruction and their strength ascertained. These arches were built of

Mannheim Portland cement, a rather poor building sand, composed of various sizes of stone, from fine sand up to round gravel stones of ½ in. diameter, and rather dusty broken trap rock, of a size that would go through a ¾-in. ring and smaller. The arches were composed of about two-thirds of coarse material, made by mixing one pail of Portland cement, two pails of sand and four pails of stone together, and about one-third of a finer material, which was made by mixing one pail of cement with two pails of sand. The cement was of such a fineness that 98.8% passed a No. 50 sieve, 92.5% a No. 74 sieve, 88.9% a No. 100 sieve, and 67.1% a No. 200 sieve. Two briquettes of three parts by weight of sand to one part of cement had a strength of 250 and 256 lbs. per square inch when 28 days old. The amount of sulphate of lime present was 2.55 per cent.

The arches were made by putting in between 12-in. I beam skewbacks three curved ribs, one in the center and one at each end, of either I or T section. These were struck to a true circle of about 8 ft. radius. The shapes were wedged to the skewback beams by means of thin steel wedges and came down as low as the fillet joining the web and flange. Below the beams was placed a board center, relatively tight, made of stripping, the center coming close against the bottom flange of the beams. Starting at one skewback a certain amount of coarse stuff was put in for about two-thirds of the thickness of the arch, and then fine stuff was filled in to the level of the top flange of the small curved beams. Then a rammer was applied tangentially to the soffit of the arch, compressing the material along the lines of the arch; the amount of compression will be seen from the tables of the tests, where the number of cubic feet of material as measured by the buckets, each ingredient being separately measured, is given, and also the cube of the arch after being rammed. First, one side was built up almost to the center, then the other side; finally the key was filled in, and rammed vertically. After the arch all the way over had been built, the haunches were filled in on the lines shown in Figs. 3 and 4 with coarse material, and then a little fine material was spread over the top in order to give a smooth and even surface. Considering the manner in which the work was done and the character of labor employed, the author thinks that it would be fair to assume that this was done with about the same care that would be employed in a building where reasonable supervision was exercised on the part of the contractor. The men who were em-

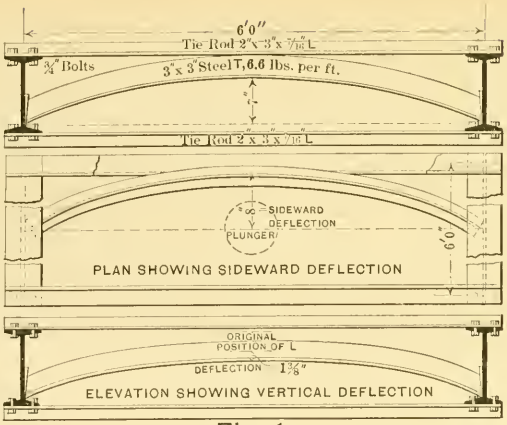


Fig. 1

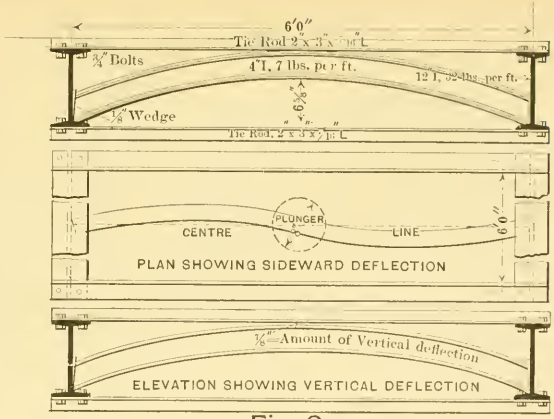


Fig. 2

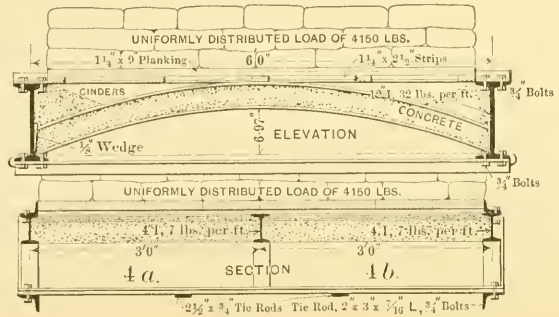


Fig. 3

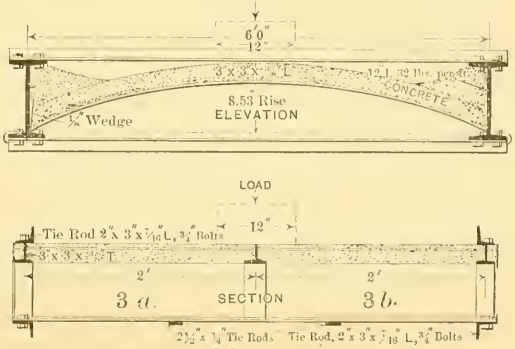


Fig. 4

ployed to do the work were not in any sense experts, and therefore could not be expected to get the very best results from the material. All of the arches were completed by October 27th, and were then permitted to set during a rather severe winter until January 25th, 1895, when the general testing began. During the time immediately after the arches were built, there was no frost to injure them, the minimum temperatures as reported by the Weather Bureau in New York being 32° Fahr. on November 7th, 9th, 11th and 15th, and 28° Fahr. on November 12th.

Test No. 76.—This was of a steel T section of ordinary commercial make, and presumably of 16 000 lbs. safe working strain, 32 000 lbs. elastic limit, and was intended to ascertain the strength of the section alone. Upon the application of the load, the crown flattened out to the position shown in Fig. 1, and then sagged off to one side, continued pumping resulting only in increasing the horizontal deflection and decreasing the pressure. Fig. 17 gives the curve of loading during this test, the maximum load being 14 000 lbs. Deflection began at 9 500 lbs.

Test No. 77.—This was of a steel I beam of the size shown, and for the same purpose as Test No. 76. Under test, the beam first flattened at the crown; then twisted to right and left so that in plan it formed an elongated letter S, as shown in Fig. 2, and then the top and bottom flanges wrinkled or buckled under the pressure, the bottom flange buckling the more. This indicated that the entire section was under compression, except perhaps a small section of the top flange near the crown. Fig. 18 gives the curve of loading during this test. The maximum load was 14 000 lbs.

Test No. 78.—This was of an arch eight days old, built as shown in Fig. 3, which was leveled up with cinders in the usual manner, but without any concrete or grout poured in. It was then stripped with $1\frac{1}{4}$ x $2\frac{1}{2}$-in. strips placed parallel with the skewback beam and set about 16 ins. on centers, and then planked with $1\frac{1}{4}$-in. plank so put on that the plank bore directly on the cinders. The arch was uniformly loaded with flagstones; each stone was measured and its weight determined by allowing 160 lbs. per cubic foot or 0.092 lb. per cubic inch. The purpose of this test was to determine how quickly the material could be used with safety as a working floor; no deflection, cracking or change of any kind in the material was observed. Table

No. 1 gives the weights per square foot, etc., while the composition of the arch in detail was as follows:

DETAILED COMPOSITION OF ARCH OF TEST NO. 78.

Arch.	Cubic feet. Coarse.	Cubic feet. Fine.	Compacts.	Cubic feet. Coarse.	Cubic feet. Fine.	Compacts.
4 *a*.....	0.908	0.908	0.833	0.908	0.681	1.675
	1.362	0.908	0.833	1.135	1.362	Key.
	0.908	0.681	0.833	1.815	.....	Leveling.
	1.362	0.908	0.791	9.760	6.243	Total.
	1.362	0.795	0.791	.....	.....	.....
4 *b*.....	0.454	0.908	1.000	0.908	0.908	0.75
	0.908	0.908	0.833	0.908	0.908	Key.
	0.908	1.135	0.666	1.815	0.908	Leveling.
	1.815	0 908	0.833	1.815	.....	.....
	1.135	0.908	0.917	10.666	7.491	Total.

Total contents of arch, 15.26 cu. ft.

GENERAL COMPOSITION OF ARCH 4 *a* AND 4 *b*.

	Coarse.	Fine.	Coarse.	Fine.	
Sand.....	6.8	6.8	3.4	3.4	Cement.
Stone.....	13.6	...	1.36	1.135	Water.

Test No. 79.—This was made of the same arch immediately after Test No. 78. The load of flagstones was practically concentrated over a length of 26 ins. of one of the 2½-in. strips; no deflection, cracking or change of any kind was observed. The center of application of the load was at the crown of the arch midway between the center and one of the side 4-in. beams.

Test No. 80.—This was of the same arch when four days older. Additional loading was put on so that the transverse strip at the crown carried 7 625 lbs. for 40 ins. of its length, and 5 320 lbs. for 26 ins. There was no appreciable deflection, sign of crack, or other disturbance noticed in the arch.

Test No. 81.—This was of a Melan arch built with ⊤ irons, as shown in Fig. 4; the pressure was continuously applied up to about 27 500 lbs., at which point the concrete separated from the inside flange of the outside ⊤. In the underside of the arch hair cracks appeared, but there was no separation along the web of the middle ⊤. During

the time of observation, pumping was discontinued, and the pressure dropped to 22 500 lbs. Then it was applied again, when the hair cracks on the top of the crown opened $\frac{1}{16}$ in. at 30 000 lbs. At 32 000 lbs. the outside T went out $\frac{3}{32}$ in., and the top surface of the concrete began to spall. At 35 000 lbs. the cracking on the haunches began. During the progress of this test, one of the 2¼ x ¾-in. tie rods pulled out on the horizontal section just below the bend. The broken end shows rather a poor character of metal failing under direct tension. The curve of loading is shown in Fig. 19. The dotted line in this figure is the curve of deflection. The composition of the arch was as follows :

General Composition of Arch of Test No. 81.

	Coarse.	Fine.	Coarse.	Fine.	
Sand	4.075	4.075	2.04	2.04	Cement.
Stone	8.150	.....	1.36	1.02	Water.

Test No. 82.—This arch was built with 4-in. I beams. The load was applied at the crown over an area of about 1 sq. ft., and midway between the center and an outside beam. The outside I beam was wedged in place laterally against the upper angle iron used to hold the I beam skewbacks in place, and therefore met the conditions usually occurring in a floor where there would be support on all sides of the points of additional loading. The load was continuously applied for five minutes until it reached 35 000 lbs., at which point there was a separation from the web of the outside beam. At 47 500 lbs., it cracked along under the beams and cracked also on top at the center. At 52 500 lbs. a second crack appeared in the center, and at 55 000 lbs. after 10.5 minutes the machine clamp broke. The section of the clamp at the point of fracture was 3 x ¾ ins., and the failure occurred at the point where it was bent to clasp the skewback beam. The cracks closed, leaving a permanent deflection of 0.22 in. In this test, the tie rods were moved together immediately under the portion of the arch attacked, so as to have as much strength there as possible. The failure of the arch had occurred, but its strength was still very great. The construction of this arch is shown in Fig. 3, and its composition in detail is given in the following table :

DETAILED COMPOSITION OF ARCH OF TEST NO. 82.

Arch.	Coarse.	Fine.	Compacts.	Arch.	Coarse.	Fine.	Compacts.
a....	0.908	0.908	.....	2 *b*....	1.360	0.908	0.940
	1.135	0.908	1.580		0.908	0.908	1.790
	0.908	1.135	2.460		0.908	0.908	2.450
	1.360	0.568	0.812		1.360	0.568	0.796
	1.360	0.794	1.540		0.908	0.798	1.560
	0.908	0.680	1.690		0.908	0.908	2.330
	1.135	1.360	.....		0.908	1.360	Against skewback and for leveling.
	1.820	.....	.....		2.100	.....	

Total contents of arch, 15.26 cu. ft.

GENERAL COMPOSITION OF ARCH 2 *a* AND 2 *b*.

	Coarse.	Fine.	Coarse.	Fine.	
Sand.....	27.90	29.8	13.950	14.89	Cement.
Stone.....	55.75	....	5.575	5.20	Water.

Test No. 83.—This was practically a continuation of Test No. 82, new machine clamps being put in position and the same blocks used at the same position on the arch for the application of the load. The arch was continuously loaded up to 52 000 lbs., requiring 13 minutes; then pumping was discontinued, and observations made, during which time the pressure dropped. Then the loading was continued up to 58 750 lbs., at which point failure was deemed to have occurred, the duration of the test being 17.5 minutes; the arch was still, however, so strong that after the removal of the outside curved I beam, it was difficult work to break it down with a sledge so as to make it fall in, and it would no doubt still have carried a considerable load. At the time of the failure, there were cracks radiating from each corner of the block used for imposing the load to the point of intersection of the enclosing I beams with the skewback beams, showing that the whole material had been crushed down.

Test No. 84.—The load was continuously applied over the crown of the center 4-in. I beam up to 42 500 lbs., when the loading was suspended for a few moments, until a new position of the deflection stake was obtained, and then applied a second time. At 55 000 lbs., the maximum load, the beams separated from the concrete, and the concrete cracked. Continued pumping resulted in no increase of the

pressure, but in an increased deflection and an opening of the cracks, until the deflection bar caught. Fig. 20 shows the curve of loading. The solid line is the curve of loads, and the dotted line is the curve of deflection, the total deflection being 0.585 in. One of the tie rods was a little slack at all times. The composition of the arch is given in the following table :

DETAILED COMPOSITION OF ARCH OF TEST NO. 84.

Arch.	Coarse.	Fine.	Compacts.	Arch.	Coarse.	Fine.	Compacts.
1 *a*.....	0.908	0.454	1.000	1 *b*.	1.816	0.908	1.084
	0.908	0.908	0.791		0.908	0.680	0 938
	1.132	1.022	1.178		0.908	1.022	1.000
	0.908	0.680	1.000		1.816	0.908	0.960
	1.360	0.680	0.875		1.360	0.908	0.916
	1.132	0.908	1.042		0.908	0.908	1.178
	0.908	0.908	Key.		0.908	0.908	Key.

Total contents of arch, 15.26 cu. ft.

GENERAL COMPOSITION OF ARCH 1 *a* AND 1 *b*.

	Coarse.	Fine.	Coarse.	Fine.	
Sand.....	8.16	4.08	4.08	4.42	Cement.
Stone.....	16.33	.....	2.49	1.815	Water.

Test No. 85.—This was of the same arch, one-half of which was the subject of Tests Nos. 82 and 83. Both the I beams were removed, so as to leave a concrete rib 3 ft. wide ; the tie rods were shifted across underneath the arch, so as to afford as good a support as practicable. The loading was placed eccentrically, 23 ins. from one skewback beam and halfway between the faces of the concrete rib; at 27 500 lbs. the north haunch, which was the one nearest the point of application of the load, cracked. At 31 250 lbs., reached in 4.3 minutes, spalling on the top began, the cracks continued to open, and increased pumping resulted in a decrease in the pressure.

Test No. 86.—This was a test of a combined concrete-iron lintel of the cross-section shown in Fig. 5. The load was steadily applied ; at 13 250 lbs. cracking began in the middle ; at 25 000 lbs. it spalled above the angle on one side near the center, and a well defined crack appeared on top, the cement sliding out at the ends past the angles ; at 32 500 lbs. the bolts which ran from angle to angle, and were imbedded in the concrete, were sheared off between the concrete and the angles, and the material spalled along the angles on both sides.

Test No. 87.—This was of a 4-in. I beam and concrete arch like that shown in Fig. 3, the load being applied eccentrically 1 ft. 7.5 ins. from one skewback, over the central arch beam. The arch was boarded over, and the space over the haunches filled with a leveling of cinders. The pressure was applied continuously until 30 000 lbs. was reached, at which point a stick lodged in the valve from the oil that was being used. Then the pressure was permitted to drop to zero, the valve cleaned, and the pressure reapplied continuously for 13.7 minutes up to 70 000 lbs., at which point the clamps at the north end broke at the bottom, tearing clean off. These clamps were 3 x 1⅛ ins. at the point of fracture, and broke where the clamp was bent to encircle the flange of the skewback beam.

Observations Concerning the Melan Arch Tests.—In the last columns of Table No. 1, certain deductions are drawn which are offered as indicating one way of interpreting results that can with propriety probably be interpreted in several different ways. The results of the column headed "per square inch of steel" are taken simply from deduced results in the case of the Tests Nos. 76 and 77, according as one or more of the sections are subjected directly to compression. It is probable that the concrete on the two sides considerably increases the strength of the material by preventing it from buckling as it did when loaded alone. It is also probable that the strength of the concrete is in a measure increased by the I beams; although the figures given are not very conclusive, since in the only case where the concrete was attacked alone it was attacked by an eccentric load, bringing into account its tensile strength, while in all of the other cases, save one, it was permitted to act almost exclusively in compression.

Tile Arch Tests.—These were tests of a series of arches, some of which were built at the works of the Staten Island Terra Cotta Lumber Company, but the majority by the Raritan Hollow and Porous Brick Company, at various times during the winter of 1894–95 and the summer of 1895.

Test No. 88.—This was made on a hollow tile end-construction arch 7½ ins. deep and of an effective depth of about 6 ins., made of hard tile and laid up without any mortar, one block wide. This arch is shown in Fig. 6. The clay blocks were fitted in place over a temporary center and then the key block put in and wedged as well as it could be, but owing to irregularities of the material this could not be very satis-

TABLE No. 1.—Melan Arch Data.

Number.	Span.	Rise	Width.	Areas.		Loads.			Deflection.	Thickness.	Material.	Areas in Compression.		Tie rods. Tension.	Ultimate shear.	Horizontal Thrust.		
				Total.	Loaded.	Total.	Per square foot of area.	Per square foot, loaded.				Steel.	Concrete.			Total.	Per square inch of material.	
																	Steel.	Concrete.
	Ft.	Ins.	Ft.	Sq. ft.	Sq. ft.	Lbs.	Lbs.	Lbs.	Ins.	Ins.		Sq. Ins.	Sq. Ins.	Lbs.	Lbs.	Lbs.	Lbs.	
76....	6	7.00	0.25	.....	.....	14 000	.....	.....	1.62	3	Steel T	1.95	.....	84 250	53 000	36 000	18 450	.....
77....	6	6.66	0.20	.....	.....	14 000	.....	.....	0.87	4	" I	2.05	.....	84 250	53 000	37 800	18 450	.....
78 ..	6	6.97	6.00	36	3.16	4 150	115	131	0	4	Steel, concrete.	6.15	212	168 500	106 000	5 360	.....	.....
79....	6	6 97	3.00	18	3.24	5 320	296	1 642	0	4	"	4.10	106	84 250	53 000	13 740	.....	.....
80....	6	6.97	6.00	36	9.90	12 945	360	1 310	0	4	"	6.15	212	168 500	106 000	33 400	.....	.....
81....	6	8.53	4.00	24	1.00	40 000	1 667	40 000	0.43	3	"	1.95	70 5	168 500	106 000	80 450	18 450	880
82....	6	7.28	3.00	18	1.25	55 000	3 055	44 000	0.47	4	"	4.10	106	168 500	53 000	135 800	36 900	933
83....	6	7.28	3.00	18	1.25	58 750	3 260	47 000	0.59	4	"	4.10	106	168 500	53 000	145 000	36 900	1 019
84....	7	7.52	6.00	36	1.00	55 000	1 528	55 000	0.58	4	"	2.05	106	168 500	106 000	153 800	18 450	1 277
85....	6	7.28	3.00	18	1.50	31 250	1 735	20 850	0.49	4	"	.....	106	84 250	53 000	87 900	.....	839
86....	.....	.....	.....	.....	.....	32 500	.....	.....	0.52	.....	"	.....	.....	.....	.....	.....	.....	.....
87....	6	7.28	6.00	36	1.00	70 000	1 945	70 000	0.10	4	"	2.05	106	168 500	106 000	180 800	18 450	1 530

Note.—In the calculations here made, the ultimate tensile strength of the tie rods is assumed to be 50 000 lbs. per square inch. The ultimate shearing strength of the rivets in single shear is taken to be 30 000 lbs. per square inch.

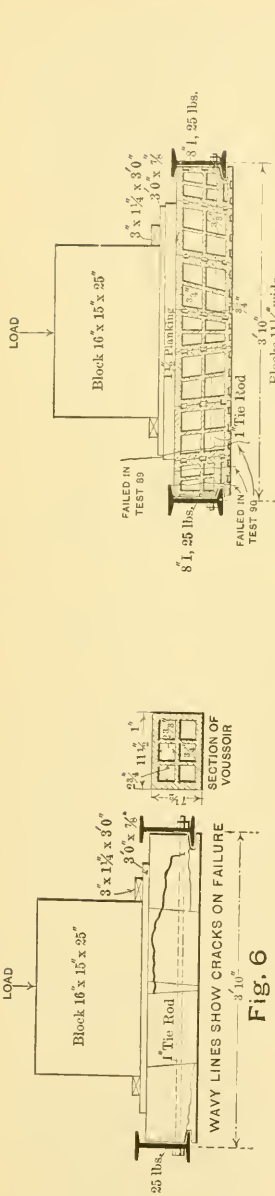


Fig. 6

Fig. 7

Fig. 9

Fig. 5

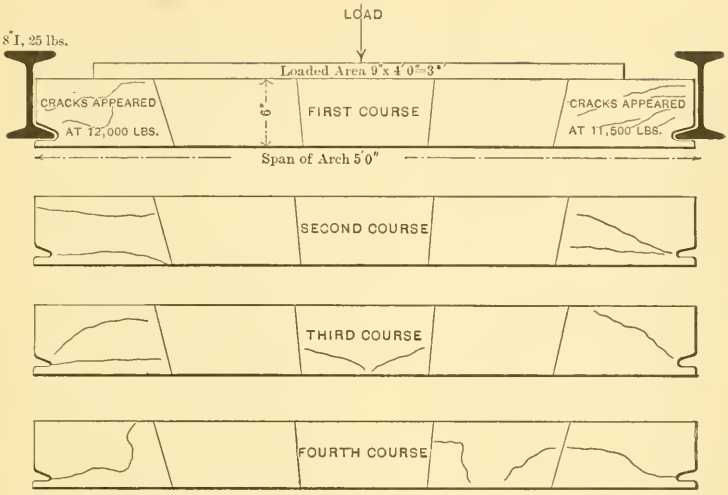
Fig. 10

factorily accomplished. The pressure was continuously applied, and at 5 000 lbs., the key at the upper portion split in a plane parallel with the tie rod. At 6 250 lbs. the skewback on the right-hand side cracked, and a spall flew off the middle voussoir on the right-hand side of the key. At 9 000 lbs. the arch failed along the wavy lines shown. The application of the load was by means of planking leveled up with sawdust over the top of the arch, so as to effect a practically uniform distribution of the loading.

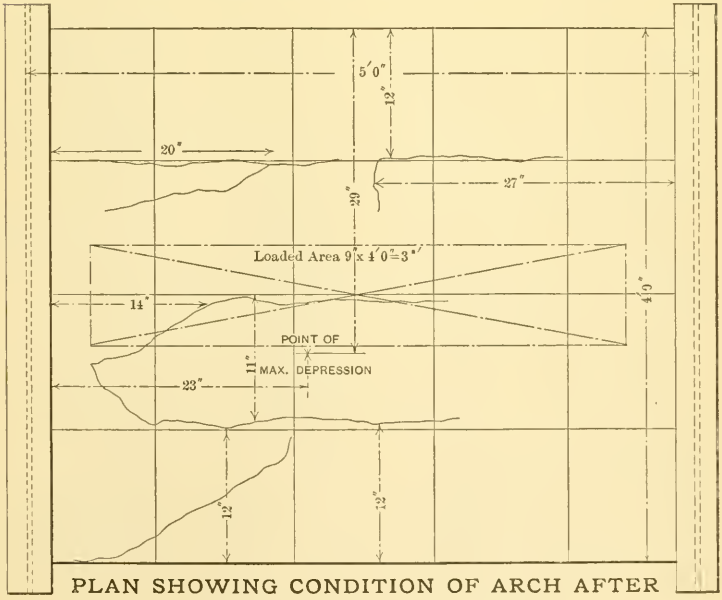
Test No. 89.—This was of a side-construction or old style arch laid up dry like No. 88, and of hard tile. Its construction is shown in Fig. 7. The pressure was applied as shown, there being a slight leveling of sawdust under the planking. There was no perceptible effect until the load reached the maximum of 2 500 lbs., when the arch suddenly collapsed by shearing off the voussoir next the skewback at the point shown. The arch was composed of one course of voussoirs laid up dry; the load was practically uniformly distributed.

Test No. 90.—This was similar in all respects to No. 89, except that in this case, not only the voussoir next the skewback, but the skewback itself, failed by shearing.

Test No. 91.—This was of a hard tile end-construction arch which had been built for some time, and had for several months carried a pile of brick weighing 12 000 lbs. uniformly distributed over it. A little sand was sprinkled over the top, and the load was applied over an area of 9 x 48 ins., or 3 sq. ft., in the center. The point of application of the load was over one of the central longitudinal divisions, and as each one of the blocks composing the arch was about 1 sq. ft. in area, two rows of blocks received the loading. As the load was applied the arch continually deflected until the maximum of 13 750 lbs. was reached, after which continued pumping resulted in a decrease in the amount of the load. The cracking of the various courses is shown in Fig. 8, which does not, however, show how markedly the top horizontal section of the arches was sheared off, showing that it acted truly in compression. There was a certain amount of elongation of the tie rods, but they remained intact. This arch had an effective depth of 4 ins.; the balance being below the skewback and acting simply as fire-proofing for the upper section. In a plane perpendicular to the line of pressure, the section through the arch would show a top horizontal and a bottom horizontal member, a middle horizontal member, and in each



ELEVATIONS SHOWING CRACKS THAT APPEARED UNDER MAXIMUM LOADING.



PLAN SHOWING CONDITION OF ARCH AFTER PRESSURE WAS REMOVED.

FIG. 8.

block three or four vertical members. The arch was laid up in a mixture of one part Portland cement and two parts sand, and over the top of it was a leveling of the same mortar about ½ in. thick; when taken off, chips of the blocks adhered to this, showing that the tile section was reinforced by about this same amount. Fig. 21 shows the load curve.

Test No. 92.—This was of an end-construction arch, nominally 9 ins. deep, and of 5 ft. 5 ins. span, 4 ft. 2 ins. wide, with an effective depth of about 7¾ ins. The tie rod was so placed as to be 2½ ins. above the bottom, thus reducing the distance from the center of the tie rod to the top of the arch to 6 ins. The section of the voussoirs, which were of hard tile, is shown in Fig. 9. Blocking was so placed over the arch as to distribute the load uniformly over it, and then the pressure was gradually applied until the machine clamp broke at 57 500 lbs. At 40 000 lbs. a slight crack appeared in the bottom of one of the skewbacks running out horizontally from the bottom. At 40 500 lbs. the bottom of the joint of the key block on the left-hand side opened ⅛ in., and on the right-hand side it opened $\frac{1}{16}$ in. At 42 500 lbs. the top began to spall a very little at the center. After the clamp broke the cracks on the arch all closed, and the only evidence that it had been subjected to load was a spall the size of a thumb-nail at the top corner of one of the blocks, and a small crack at the skew in one of the others. This arch was laid in Portland cement mortar, and had set 24 hours when tested, but the temperature had been so low that the mortar had frozen.

Tests of Arches Built for the Purpose.—Table No. 2 gives the weight and composition of these arches. They were laid up in a strong Portland cement and had about three months in which to set.

TABLE No. 2.—Special Arches for Testing.

Test Number.	93	94	95	96	97	98	99	100	101
Depth of block, inches	8	8	8	8	10	10	12	8	8
Weight of block alone, per square foot, pounds	31.3	32.1	36.1	32.5	41.6	40.3	45.0	34.8	32.6
Weight of arch, per square foot, pounds	42.6	42.3	47.4	44	57.5	53.8	57.4	47.0	42.7
Weight of blocks, total, pounds	469.25	481.5	531.5	493.25	624	620.75	769.5	550	514
Weight of sand, pounds	86	78	83	88	116	102	110	96	82
Weight of cement, pounds	68	60	66	70	97	84	82	77	63
Weight of water, pounds	32	30	32	34	49	42	44	38	32
Total weight, pounds	655.25	649.5	712.5	685.25	886	848.75	1 005.5	761	691
Total weight, less half water, pounds	639.25	634.5	696.5	668.25	861.5	827.75	983.5	742	675

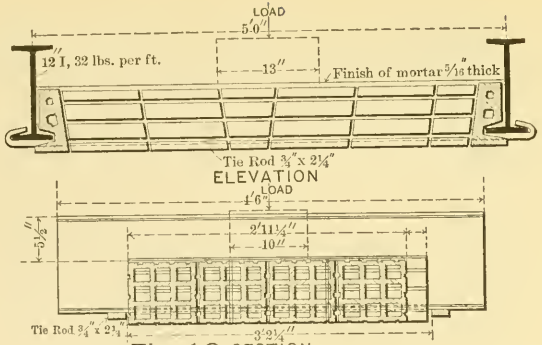


Fig. 12 SECTION

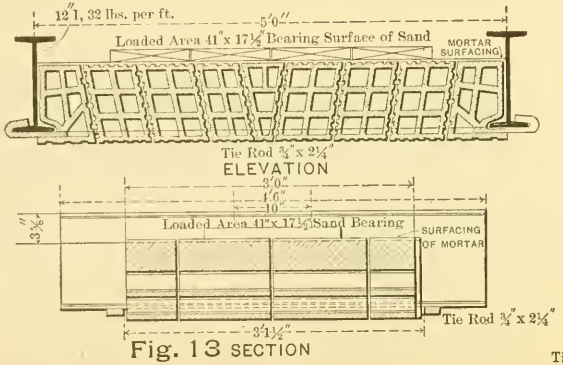


Fig. 13 SECTION

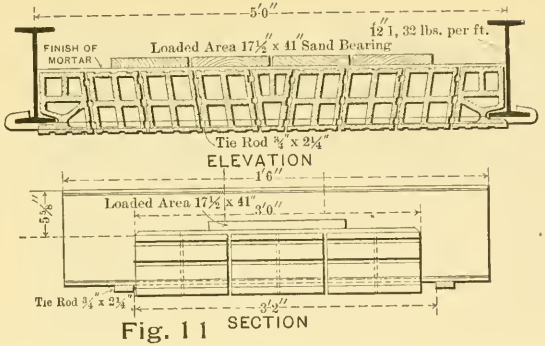


Fig. 11 SECTION

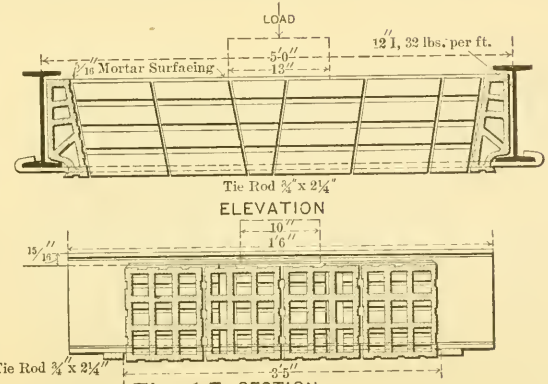


Fig. 15 SECTION

Test No. 93.—This was a hard tile side-construction arch with 8-in. blocks, the effective height of the arch being 7 ins. It was laid in Portland cement with a leveling of about $\frac{5}{16}$ in. of cement on top; the load was applied near the center on an area of 10 x 12 ins. (see Fig. 10). The tying was done by hook rods placed as shown. At 5 250 lbs. the first crack appeared, and the key opened about $\frac{1}{16}$ in., at 8 750 lbs. the crack opened wider, and cracks appeared at the left skewback, opening wide; at 10 000 lbs. the arch collapsed, and cracks appeared at the left skewback; there was a crushing in of the voussoir along its entire length under the right-hand edge of the block on which the load was applied.

Test No. 94.—This was of a side-construction hard tile arch, shown in Fig. 11. The loaded area was $7\frac{1}{2}$ x 41 ins. The load was continuously applied; at 5 000 lbs. there was a slight opening of the skewback; at 5 700 lbs. there was a failure of the east skewback by shearing. Fig. 1, Plate IX, shows the nature of the failure.

Test No. 95.—This was of an arch of hard tile end construction, with porous terra cotta solid skewbacks; the effective height of the arch was $5\frac{3}{4}$ ins., although the blocks were nominally 8 ins. There was $\frac{5}{16}$ in. finish of mortar on the top, and the load was applied on an area of 10 x 13 ins., as shown in Fig. 12. At 5 000 lbs. there was a slight hair crack at the junction of the skewback block with the skewback I beam, and a slight crack at the right-hand side and bottom of the key block. There also appeared a slight hair crack in the mortar cover of the top of the arch. At 6 250 lbs. the crack on the right-hand side of the bottom key opened $\frac{3}{32}$ in., and the top surfacing began to crack. At 7 500 lbs. it cracked in the middle and rose, and the crack in the key opened $\frac{1}{8}$ in. At 8 750 lbs. the top cover bulged further, and the arch opened $\frac{3}{8}$ in. at the bottom of the key. At 10 000 lbs. the top surfacing had bulged 1 in., and the plaster began dropping out (see Fig. 3, Plate IX). At the left-hand end of the arch, the joint between the skewback block and the end construction block opened, and at 11 250 lbs., the maximum load, the arch practically failed, the top cover bulging and cracks radiating out in it from each corner of the block on which the pressure was applied, at an angle of about 45° with lines drawn perpendicular and parallel to the skewback beams. The pumping was renewed, when the bottom of the skew dropped off at the left-hand end, being sheared

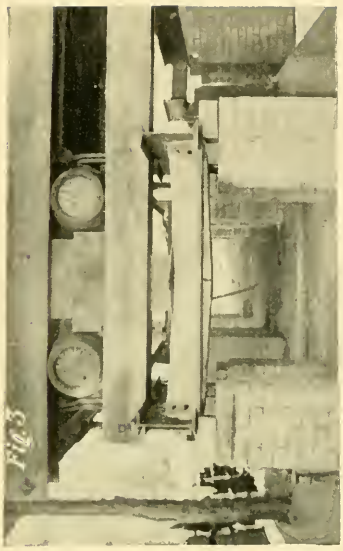


Fig. 3

Fig. 4

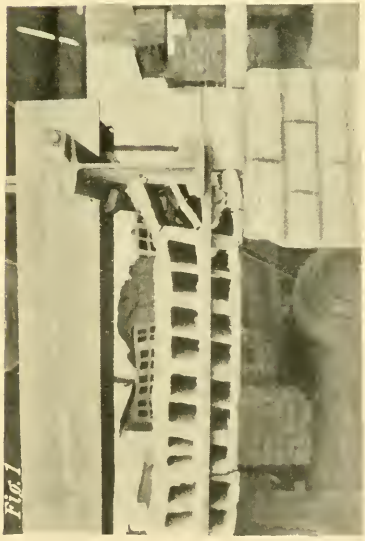


Fig. 1

Fig. 2

through. Then the two east skews, or the parts of the skews which protected the bottom flange of the I, dropped off, and a spall appeared in the bottom of the first voussoir away from the skew, at the left-hand end. At the right-hand end the flange protection dropped off, and cracks appeared running to the right and left from the upper half of the lower hole in the skewback block. Fig. 22 shows the curve of loads for this test and for No. 96.

Test No. 96.—This was of an end-construction arch similar to Test No. 95, and shown in Fig. 12. At 5 000 lbs. cracks appeared at the bottom on the right-hand side of the key. At 10 000 lbs. these cracks opened $\frac{1}{8}$ in.; the top surface bulged up. At 12 500 lbs. the cracks at the bottom of the key opened $\frac{1}{4}$ in.; cracks appeared between the skewback block and the skewback beam along the left-hand end, and a crack also appeared along the left-hand end in the skew and the first voussoir just above the bottom fillet of the web of the skewback beam, as shown in Fig. 2, Plate IX. This skew then crushed off over the whole bottom, and the facing plaster dropped off from the top. The maximum load was 13 000 lbs. The key was cracked along the top cover plate, and the skew was a splintered mass.

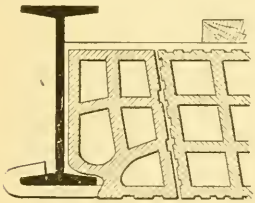


Fig. 14.

Test No. 97.—This was of a side-construction arch of the section shown in Fig. 13, with the load applied over an area of 41 x 17½ ins. The effective depth of the arch was 7½ ins., and there was a leveling of mortar on the top. The key course was quite high in the center so that the load was applied almost directly on it. The course next west of it was also a little high. At 5 000 lbs. a crack opened at the bottom of the key on the left-hand side about $\frac{1}{16}$ in. At 10 000 lbs. failure occurred at the top of the voussoir, the third one from the left-hand skewback, or the second one from the key. All through that course there was a crushing of the top web. The flange protection of the skewback also sheared off.

Test No. 98.—This was of a side-construction arch with a similar loading, namely 17½ x 41 ins. The skewback of this arch is shown in Fig. 14; the remaining details are indicated in Fig. 13. At 5 000 lbs. a crack appeared on the left-hand side of the key at the bottom, and at 12 500 lbs. the top cover began to break up, while failure occurred at 14 500 lbs., the web along the voussoirs next to the skewbacks at

the left-hand end shearing all across and directly down. The effective rise of this arch was 8 ins.

Test No. 99.—This was of an end-construction arch nominally 12 ins. deep, having an effective rise of 10 ins., and with a side-construction skewback of hard tile, shown in Fig. 15. At 3 750 lbs., reached in one minute, a hair crack appeared on the right-hand side of the key, and showed through to the other face. At 7 500 lbs., reached in 2.5 minutes, the skewback began to show failure at both sides. At 10 000 lbs. no change took place. At 11 250 lbs., reached in 4 minutes, the arch again began to fail at the skewbacks, and the crack next to the key at the bottom opened $\frac{1}{16}$ in. At 15 000 lbs., reached in 13.2 minutes, the cracks showed all along the skewback joints on both sides. At 16 400 lbs., reached in 14.3 minutes, the arch failed, the top surface cracking, a slight crack appearing in the top cover of the key, and the skews cracking off just below the diagonally inclined piece on both sides, the bottom cover piece falling off (see Fig. 4, Plate IX).

Test No. 100.—This was of an end-construction arch, similar to that shown in Fig. 12, except the skewbacks, which are shown in Fig. 16, and were of the porous terra cotta solid type; the effective rise was 7 ins., and there was a slight finish of mortar on top. The load was applied on an area of 10 x 13 ins. At 2 500 lbs., after 1 minute loading, a hair crack appeared on the right-hand side of the bottom key. At 5 000 lbs., after 1.5 minutes, a crack appeared on the east skewback by the block moving away from its beam; at 6 500 lbs., after 2 minutes, the crack at the bottom of the key widened to $\frac{1}{16}$ in.; at 7 250 lbs., after 2.3 minutes, the left-hand skewback cracked on the top away from the first voussoir; at 8 000 lbs. the same thing happened with the right-hand skew; at 9 000 lbs., after 3 minutes, the cement cover on top of the key began to rise away from the surface. At 10 000 lbs. this began to crack over the top of the key; at 12 000 lbs. the left-hand skewback began to crack where the lower hole went through it. At 12 500 lbs. the top surface on the key cracked very badly, and the cracks at the skewbacks opened $\frac{1}{4}$ in. each. At 13 500 lbs., after 5.5 minutes, the key at the bottom cracked open $\frac{3}{4}$ in., and the bottom plaster began to fall out, and at failure the point of the left-hand skew sheared off, the key cracked through the middle, near the top section opening about $\frac{1}{2}$

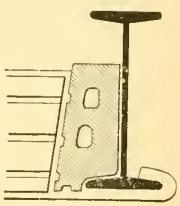


Fig. 16.

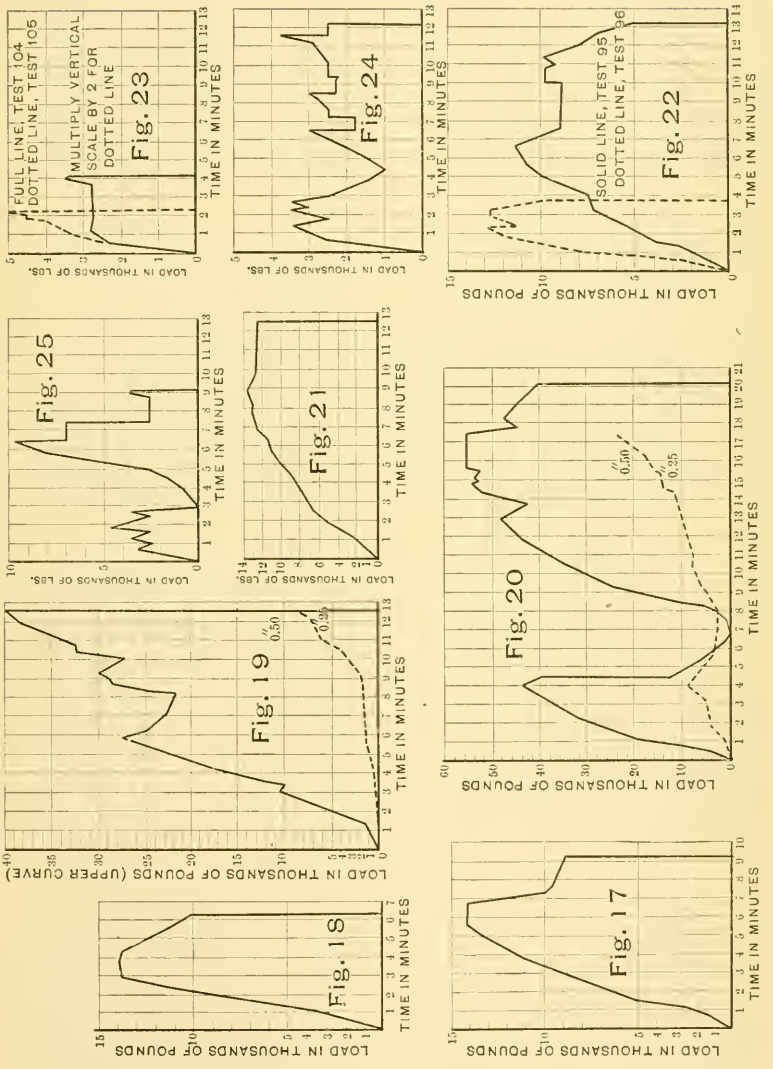


Fig. 17
Fig. 18
Fig. 19
Fig. 20
Fig. 21
Fig. 22
Fig. 23
Fig. 24
Fig. 25
SOLID LINE, TEST 95
DOTTED LINE, TEST 96
FULL LINE, TEST 104
DOTTED LINE, TEST 105
MULTIPLY VERTICAL SCALE BY 2 FOR DOTTED LINE
LOAD IN THOUSANDS OF POUNDS
LOAD IN THOUSANDS OF POUNDS (UPPER CURVE)
LOAD IN THOUSANDS OF LBS.
TIME IN MINUTES

in., and near the bottom section a crack opened that was just perceptible.

Test No. 101.—This arch was similar to the last. At 2 250 lbs. a crack opened on the right-hand side of the bottom key; at 10 000 lbs. this crack opened $\frac{3}{32}$ in.; the top surface of mortar began to spall up. Cracks appeared between the skew and the first voussoir on the left-hand side. At 13 000 lbs. the crack in the bottom of the key opened $\frac{1}{8}$ in., and the cement cover began rising on top in the center. At 15 000 lbs., after four minutes, the crack at the center began to spall, the plaster began falling off at the bottom, and the key began to splinter. When failure occurred the arch was all splintered on the top, and when the top was taken off it was found to have cracked along the line of the arch thrust. The maximum load was 15 750 lbs. The left-hand skew cracked between the two holes, shearing out and dropping off. The first voussoir from the right-hand skew had the bottom plate sheared off entirely, all of the cracks showing the true arch action.

All of the following tests were made with hard tile arches laid up without any mortar. A little damp sand was used to bed the skew-backs of the side-construction arches and to even up the top for the first plank. They were all built between the same pair of beams and usually sagged a little after the center was struck.

Test No. 102.—This was of a hard tile end-construction arch, with a side-construction key, the load being placed as shown in Fig. 1, Plate X, on the top of the key. Immediately on the loading being started, there was a pinching in of the key and cracking at the point where the top of the voussoirs struck it; but the key section of $\frac{3}{4}$-in. material stood under the vertical load until a maximum of 3 100 lbs. was reached after 2.2 minutes. Data concerning this arch is given in Table No. 3, page 567.

Test No. 103.—This arch was 12-in. end-construction protection-skew hard tile; the key block had the corners chipped out as shown in Fig. 5, Plate X; the left skew was the only one that failed; the right-hand voussoir which had the numbering on cracked on top as shown. The maximum load of 5 000 lbs. was reached in 2.2 minutes.

Test No. 104.—This arch was of the side-construction protection-skew hard tile block type and failed under a distributed load made as shown in Fig. 2, Plate X, which also shows the manner of the failure.

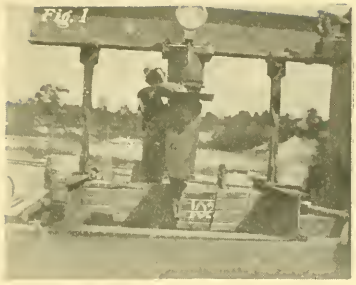


Fig. 1

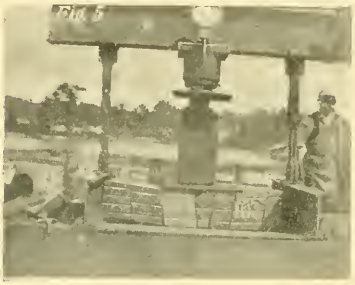


Fig. 2

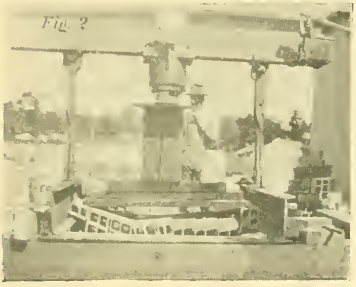


Fig. 3

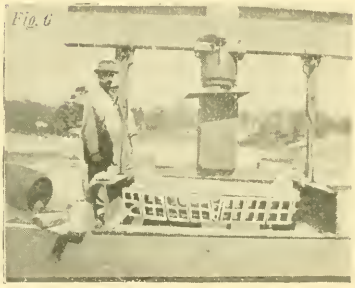


Fig. 6

Fig. 5

Fig. 7

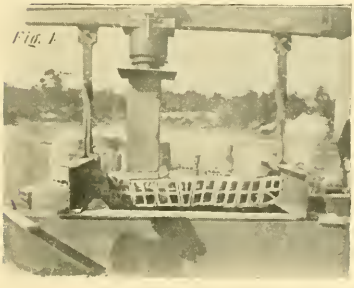


Fig. 4

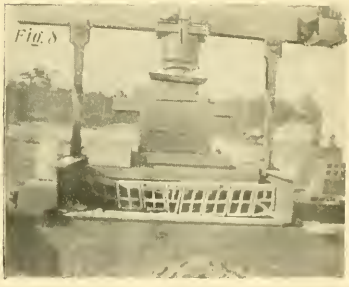


Fig. 8

The only block which failed was the left-hand skew, which sheared off by reason of the thrust of the diagonal piece, not being supported by the flange of the beam. The maximum pressure in this case, 3 500 lbs., was reached in 4 minutes.

Test No. 105.—This arch was built of the same voussoir blocks as Test No. 104, with a new key block and with plain skewbacks, as shown in Fig. No. 6, Plate X. The result was that the arch carried a load of 10 000 lbs., and, when failure occurred, the left-hand skew broke all to pieces. The curve of loading of Tests Nos. 104 and 105 is shown in Fig. 23.

Test No. 106.—This arch was of the 12-in. end-construction hard tile protection-skew type, loaded over 10 minutes, the maximum being reached at 4.5 minutes; the crown deflected about 1½ ins., due to the crushing in of the material. At the beginning the joint at the bottom of the key was open ¾ in. On the side opposite to that shown in Fig. 3, Plate X, it was practically intact, and, at the time the photograph was taken from which the cut was made, the arch was carrying over 8 000 lbs.

Test No. 107.—This was of a 12-in. end-construction hard tile arch, which began to spall on the top at 2 250 lbs.; the west or left-hand skew split at 2 500 lbs., and the voussoir at the right-hand side of the key split at the top. This arch is shown in Fig. 7, Plate X.

Test No. 108.—This was of a 12-in. end-construction hard tile arch, similar to the preceding, eccentrically loaded, the center of the load being placed 18 ins. from the center of the web of one skewback beam. When the load was placed on this arch the voussoirs at the left-hand side under the point of application of the load slipped past the key block, fracturing and splintering it. Increased pumping resulted in no increase in the pressure. The maximum load was 3 800 lbs. After it was found that no gain in pressure could be made and that the plunger had gone almost the full length of its travel, the load was discontinued and many of the voussoir blocks were found to be intact, except for splintering on the edges. The curve of loads is shown in Fig. 24.

Test No. 109.—This arch was of the 8-in. side-construction hard tile plain skewback type. It was loaded eccentrically with the center of gravity of the load placed 18 ins. to the right from the left skewback beam, and failure occurred by the shearing of the top web at the

left-hand edge of the load block, as is shown in Fig. 4, Plate X. The maximum load of 1 500 lbs. was reached in 1.2 minutes.

Test No. 110.—This arch was similar to No. 109, but centrally loaded. Failure occurred by the collapse of the voussoir next to the key at the left-hand end, or under the edge of the point of application of the load. The balance of the arch remained intact. The maximum load of 2 500 lbs. was reached in 2.7 minutes.

Test No. 111.—This was made on an arch two courses wide, of blocks similar to those of Nos. 109 and 110, with about 1½ ins. of sand covering them; the bearing blocks were 18 ins. wide, 4 ft. 4 ins. long; the lower edge of one skewback broke off at 9 950 lbs. Continued pumping resulted in a decrease in the pressure to 2 500 lbs., and then the second course tumbled in. Fig. 8, Plate X, shows how the left-hand skewback failed by shearing down at the point of junction of the curved rib with the vertical and horizontal portions. This skew was the only part of the course that was injured, which can be seen in the illustration. The curve of loads is shown in Fig. 25.

Test No. 112.—This was similar to No. 111 in all particulars; the loading was on a plank 10 ins. wide and 2 ft. long, centrally applied. At 2 000 lbs. the bottom began to open and there was a little crushing at the crown; at 2 500 lbs., reached in 3.4 minutes, the voussoirs next to the point of application of the load to the right and the one north of it both crushed in, letting down both courses.

Test No. 113.—This test was made on two courses arranged as in Test No. 112, and eccentrically loaded. At 1 250 lbs. one of the voussoirs east of the west skewback cracked through on the edge near the top. At 2 000 lbs. it was cracking off in slivers. When failure occurred, the top voussoir collapsed just to the right of the left skewback. The voussoir immediately under the load also fell in at the point of fracture. The maximum pressure was 2 750 lbs., reached in 2.5 minutes.

It is evident from Table No. 3 that there are not a sufficient number of tests recorded here from which to draw many general conclusions other perhaps than the following:

First.—That the side-construction arch requires a skew in which the inclined member on the line of the arch thrust runs back to a solid support against either the fillet or the flange of the skewback beam.

Second.—The end-construction arch requires good mortar and

TABLE No. 3.—Data of Clay Arch Tests.

Test number.	Span. Inches.	Rise. Inches.	Width. Inches.	Total arch area. Square feet.	Loaded arch area. Square feet.	Total load. Pounds.	Load per square foot of arch. Pounds.	Load per square foot loaded. Pounds.	Area tie rod. Square inches.	Horizontal thrust. Pounds.	Shear per foot skew. Pounds.	Depth of skew. Inches.	Blocks. Style.	Blocks. Material.	Blocks. Skew.	Load.	Remarks.
88	46	5	11.5	3.67	2.88	9 000	2 455	3 125	1.57	10 340	4 670	7.5	End	Hard	Protection	Distributed	Failed on arch lines.
89	46	5.5	11.5	3.67	2.88	2 500	682	868	1.57	2 615	1 303	8	Side	"	"	"	Sheared. Laid dry.
90	45	5	11.5	3.62	2.88	1 300	359	452	1.57	1 464	678	8	"	"	"	"	" "
91	60	3.5	48	20.00	3.00	13 750	688	4 590	1.57	29 500	1 720	6	End	"	"	"	Crushed on arch lines. Old arch in Portland cement.
92	65	6	50	22.6	20	57 500	2 545	2 875	3.52	78 000	6 980	10	"	"	Plain	"	Unbroken. Laid in Portland and frozen.
93	60	6	36	15	0.82	10 000	668	12 200	3.38	25 000	1 670	8	Side	"	"	Center	Sheared by edge of load. Portland.
94	60	5	36	15	4.98	5 700	381	1 143	3.38	8 550	950	8	"	"	Protection	Distributed	Sheared by skewback. Portland.
95	60	5	35 2	14.7	0.88	11 250	767	12 800	3.38	33 750	1 890	7.5	End	"	"	Center	Crushed porous terra cotta skew. Portland.
96	60	5	36.5	15.2	0.88	13 000	857	14 800	3.38	39 000	2 135	7.5	"	Porous T. C.	"	"	Crushed porous terra cotta skew. Portland.
97	60	7.5	36	15	4.98	13 500	900	2 710	3.38	13 500	2 250	10	Side	Hard	"	Distributed	Crushed on top by key. Portland.
98	60	8	37	15.4	4.98	14 500	943	2 915	3.38	13 600	2 360	10	"	"	Plain	"	.....
99	60	10	41	17.1	0.88	16 400	960	18 670	3.38	24 600	2 395	12	End	"	"	Center	Side skew. Skew failed. Portland.
100	60	7	38.25	15.8	0.88	14 500	919	16 500	3.38	31 100	2 290	8	"	Porous T. C.	"	"	Porous T. C Skew. Portland.
101	60	7	38.25	15.8	0.88	15 750	997	17 000	10.14	33 800	2 480	8	"	Hard	"	"	Porous T. C. Skew.
102	60	8.75	10	4.2	0.82	3 100	739	3 780	10.14	5 320	1 890	12	"	"	Protection	"	End skew, side key. No mortar.
103	60	9	10	4.2	0.82	5 000	1 190	6 100	10.14	8 350	3 050	12	"	"	"	"	All end. No mortar.
104	60	5	12	5	1.00	3 500	654	3 500	10.14	10 500	1 750	8	Side	"	"	Distributed	No mortar.
105	60	5.5	12	5	4.0	10 000	2 000	2 500	10.14	13 640	5 000	8	"	"	Plain	"	No mortar.
106	60	9	10	4.16	3.35	15 100	3 630	4 510	10.14	12 560	9 210	12	End	"	Protection	"	Deflection 1½ ins. by crushing in. No mortar.
107	60	9.5	10	4.16	1.00	2 500	601	2 500	10.14	3 950	1 525	12	"	"	"	Center	.....
108	60	9.5	10	4 16	1.00	3 000	723	3 000	10.14	3 980	2 400	12	"	"	"	Eccentric	18 ins. from W. skewback beam. No mortar.
109	60	5.5	12	5.0	1.00	1 500	300	1 500	10.14	3 440	1 750	8	Side	"	Plain	"	18 ins. from W. skewback beam, No mortar.
110	60	5.5	12	5.00	1.00	2 500	500	2 500	10.14	6 850	1 250	8	"	"	"	Center	.....
111	60	5.5	24	10	6.00	9 950	995	1 658	10.14	22 650	2 486	8	"	"	"	Distributed	Two blocks wide. No mortar.
112	60	5.5	24	10	2	2 500	250	1 250	10.14	6 820	625	8	"	"	"	Center	" "
113	60	5.5	24	10	2	2 750	275	1 375	10.14	7 190	1 100	8	"	"	"	Eccentric	Two blocks wide. 18 ins. from W. beam. No mortar.

reasonably good bedding to make the joints which transmit the pressure adequate to their duty, the weak point being the mortar in the end joints.

Third.—When well bedded, the end-construction arch is very much the stronger.

TRANSACTIONS

OF THE

American Society of Civil Engineers.

INDEX.

VOLUME XXXIV.

JULY TO DECEMBER, 1895.

NOTE.—In this Index, Subjects and Writers are given Alphabetically. Titles of Papers are enclosed by quotation marks when indexed alone or under the Author's name. When indexed under the name of a person discussing a paper, the titles are not so enclosed. Titles of Papers are entered with reference to the subject and also under the first significant word.

INDEX.

VOLUME XXXIV,

JULY TO DECEMBER, 1895.

ABBOT, F. V.
Lonesome Valley Viaduct, K., C. G. & L. R. R., 253.
Mexico-United States Boundary, 279.

ALLEN, KENNETH.
High Masonry Dams, 512.
Mexico-United States Boundary Survey, 280.
Separate Sewer System, 228.

ARCHES, FLOOR.
Tests of— Fr. von Emperger, 521; George Hill, 542.

BATES, ONWARD. Life of an Iron Railroad Bridge, 312.

BOGART, JOHN. Water-Works of Syracuse, N. Y., 58.

BRACKETT, DEXTER.
"Consumption and Waste of Water," 185, 221.
Moving 36-In. Water Mains, 538.

BRIDGES.
"What is the Life of an Iron Railroad Bridge?" (With Discussion.) J. E. Greiner, 294.
"Construction of Substructure for Lonesome Valley Viaduct, Knoxville, Cumberland Gap and Louisville Railroad." (With Discussion.) Gustave R. Tuska, 247.

BROOMALL, C. M. High Masonry Dams, 514.

BUCK, L. L. Water-Works of Syracuse, N. Y., 62.

CAMPBELL, H. H. "The Physical Qualities of Acid Open-Hearth Nickel Steel as Compared with Carbon Steel of Similar Tensile Strength," 285.

CARTWRIGHT, ROBERT. Consumption and Waste of Water, 213.

CAST-IRON.
"Notes on Manufacture and Properties of Malleable Cast-Iron." H. R. Stanford, 1.

CHRISTIAN, G. L. Water-Works of Syracuse, N. Y., 62.

CLARKE, T. C.
High Masonry Dams, 506.
Life of an Iron Railroad Bridge, 309.

COLD STORAGE.
At Reading Terminal Station, Philadelphia. Joseph M. Wilson, 159.

CONCRETE.
Use of — in Construction of Substructure for Lonesome Valley Viaduct, Knoxville, Cumberland Gap and Louisville Railroad. Gustave R. Tuska, 247.

"CONSTRUCTION OF SUBSTRUCTURE FOR LONESOME VALley Viaduct, Knoxville, Cumberland Gap and Louisville Railroad." Gustave R. Tuska, 247.
Discussion: George S. Morison, 252; Charles E. Emery, 252; Foster Crowell, 252; F. V. Abbot, 253; Bernard R. Green, 253.
Correspondence: William Barclay Parsons, 254; George H. Pegram, 255; Walter H. Gahagan, 256; Gustave R. Tuska, 258.

"CONSUMPTION AND WASTE OF WATER." Dexter Brackett, 185.
Discussion: Desmond FitzGerald, 204; L. Frederick Rice, 204; Charles G. Darrach, 204; H. F. Dunham, 204; John Thomson, 205; J. J. R. Croes, 207; E. Kuichling, 208.
Correspondence: L. J. Le Conte, 211; Clemens Herschel, 212; Robert Cartwright, 213; Daniel W. Mead, 215; Allen Hazen, 218; Dexter Brackett, 221.

CROES, J. J. R.
Consumption and Waste of Water, 207.
Water-Works of Syracuse, N. Y., 54, 57.

CROWELL, Foster. Lonesome Valley Viaduct, K., C. G. and L. R. R., 252.

DAMS.
Construction of Crib — T. C. Clarke, 507.
Masonry Core Wall in Earthen — E. Sherman Gould, 56.
"Notes on High Masonry —" John Van Buren, 493.

DARRACH, C. G.
Consumption and Waste of Water, 204.
Separate Sewer System, 226.
The Philadelphia and Reading Terminal Railroad and Station in Philadelphia, 180.
The Temperature of Lakes, 110.

"DISCHARGE OF THE MISSISSIPPI RIVER." William Starling, 347.

DUNHAM, H. F.
Consumption and Waste of Water, 204.
High Masonry Dams, 517.

EMERY, Charles E.
Lonesome Valley Viaduct, K., C. G. & L. R. R., 252.
Water-Works of Syracuse, N. Y., 60.

EMPERGER, Fr. von. "Hollow Tile Floors, Past and Present," 521.

"EXPERIMENTS ON THE DISCHARGE OF A 30-IN. STOP-Valve." J. Waldo Smith, 235.
Discussion: E. Kuichling, 243.
Correspondence: J. Waldo Smith, 246.

FANNING, J. T. The Temperature of Lakes, 67.

FIRE-PROOF CONSTRUCTION.
"Tests of Fire-Proof Flooring Material." George Hill, 542.

FITZGERALD, Desmond.
Consumption and Waste of Water, 204.
"The Temperature of Lakes," 67.

FLOORS. TILE —
"Hollow Tile Floors, Past and Present." Fr. von Emperger, 521.
"Tests of Fire-Proof Flooring Material." George Hill, 542.

FLUSH TANKS.
"Separate Sewer System Without Automatic Flush Tanks." (With Discussion.) F. S. Odell, 223.

FOLWELL, A. Prescott. Separate Sewer System, 230.

GAHAGAN, Walter H. Lonesome Valley Viaduct, K., C. G. & L. R. R., 256.

GATE.
Experiments on the Discharge of a 30-In. Water — J. Waldo Smith, 235.

GOULD, E. Sherman. Water-Works of Syracuse, N. Y., 56.

GREEN, Bernard R. Lonesome Valley Viaduct, K., C. G. & L. R. R., 253.

GREENE, J. N. Water-Works of Syracuse, N. Y., 61.

GREINER, J. E. "What is the Life of an Iron Railroad Bridge?" (With Discussion.) 294, 335.

HAZEN, Allen. Consumption and Waste of Water, 218.

HAZLEHURST, G. B. Life of an Iron Railroad Bridge, 315.

HERING, Rudolph. Separate Sewer System, 226.

HERSCHEL, Clemens. Consumption and Waste of Water, 212.

HILL, George. "Tests of Fire-Proof Flooring Material," 542.

HILL, John W. Water-Works of Syracuse, N. Y., 63.

HILL, William R. "The Water-Works of Syracuse, N. Y.," 23, 64.

HOECH, Theod. G. Life of an Iron Railroad Bridge, 327.

"HOLLOW TILE FLOORS, PAST AND PRESENT." Fr. von Emperger, 521.

HYDRAULICS.

"Discharge of the Mississippi River." William Starling, 347.

"Experiments on the Discharge of a 30-In. Stop Valve." J. Waldo Smith, 235.

IRON. CAST. See Cast-Iron.

JOHNSON, J. B. Mexico-United States Boundary, 283.

KATTÉ, Walter. Life of an Iron Railroad Bridge, 318.

KIERSTED, Wynkoop. Separate Sewer System, 227.

KUICHLING, E.

Consumption and Waste of Water, 208.

Discharge of a 30-In. Stop Valve, 243.

LAKES.

"The Temperature of Lakes." Desmond FitzGerald, 67.

LANDRETH, William B.

Mexico-United States Boundary, 281.

Separate Sewer System, 231.

Le CONTE, L. J.

Consumption and Waste of Water, 211.

High Masonry Dams, 515.

Life of an Iron Railroad Bridge, 318.

Separate Sewer System, 232.

Water-Works of Syracuse, N. Y., 64.

LINDENTHAL, Gustav. High Masonry Dams, 505.

MEAD, Daniel W. Consumption and Waste of Water, 215.

METERS.

Current— William Starling, 365.

Water— "Consumption and Waste of Water." (With Discussion.) Dexter Brackett, 185.

MEXICO-UNITED STATES BOUNDARY. Topography on the Survey of— J. L. Van Ornum, 259.

MISSISSIPPI RIVER.

"The Discharge of the—" William Starling, 347.

MONTFORT, R. Life of an Iron Railroad Bridge, 323.

MOORE, E. C. "Moving Two 36-In. Water Mains Without Shutting Off the Water." (With Discussion.) 532, 540.

MORISON, George S.
Life of an Iron Railroad Bridge, 308.
Lonesome Valley Viaduct, K., C. G. & L. R. R., 252.

"MOVING TWO 36-IN. WATER MAINS WITHOUT SHUTTING Off the Water." E. C. Moore, 532.
Discussion: Edward Wegmann, Jr., 536; James Owen, 537; Joseph C. Platt, 537.
Correspondence: Dexter Brackett, 538; E. C. Moore, 540.

"NOTES ON HIGH MASONRY DAMS." John D. Van Buren, 493.
Discussion: William E. Worthen, 505; Gustav Lindenthal, 505; Robert Ridgway, 505; F. W. Skinner, 505; T. C. Clarke, 506; Edward Wegmann, Jr., 508.
Correspondence: Kenneth Allen, 512; James Duane, 513; C. M. Broomall, 514; M. S. Parker, 515; L. J. Le Conte, 515; H. F. Dunham, 517; John D. Van Buren, 517.

"NOTES ON MANUFACTURE AND PROPERTIES OF MALLEable Cast-Iron." H. R. Stanford, 1.

ODELL, F. S. "The Separate Sewer System Without Automatic Flush Tanks," 223, 233.

OWEN, James. Moving 36-In. Water Mains, 537.

PARKER, M. S. High Masonry Dams. 515.

PARSONS, William Barclay. Lonesome Valley Viaduct, K., C. G. & L. R. R., 254.

PEGRAM, George H.
Life of an Iron Railroad Bridge, 320.
Lonesome Valley Viaduct, K., C. G. & L. R. R., 255.

"PHILADELPHIA AND READING TERMINAL RAILROAD AND Station in Philadelphia." Joseph M. Wilson, 115.
Discussion: Charles G. Darrach, 180.
Correspondence: Samuel Tobias Wagner, 181; Joseph M. Wilson; 184.

"PHYSICAL QUALITIES OF ACID OPEN-HEARTH NICKEL Steel as Compared with Carbon Steel of Similar Tensile Strength." H. H. Campbell, 285.

PLATT, Joseph C. Moving 36-In. Water Mains, 537.

POWER PLANT.
At Reading Terminal Station, Philadelphia. Joseph M. Wilson, 140.

PURDON, C. D. Life of an Iron Railroad Bridge, 314.

RAILROADS.

Knoxville, Cumberland Gap and Louisville Railroad, Lonesome Valley Viaduct. Gustave R. Tuska, 247.

Philadelphia and Reading Terminal Railroad. Joseph M. Wilson, 115.

RAILROADS, STATIONS.

The Philadelphia and Reading Terminal Railroad and Station in Philadelphia. Joseph M. Wilson, 115.

RESERVOIRS.

Temperature in— Desmond FitzGerald, 67.

RICE, L. Frederick. Consumption and Waste of Water, 204.

RIDGWAY, Robert. High Masonry Dams, 505.

RIVERS.

Measurement of Velocity in— William Starling, 363.

"The Discharge of the Mississippi River." William Starling, 347.

SCHNEIDER, C. C. Life of an Iron Railroad Bridge, 328.

"SEPARATE SEWER SYSTEM WITHOUT AUTOMATIC FLUSH Tanks." F. S. Odell, 223.

Discussion: C. G. Darrach, 226; Rudolph Hering, 226.

Correspondence: Wynkoop Kiersted, 227; Kenneth Allen, 228; A. Prescott Folwell, 230; William B. Landreth, 231; L. J. Le Conte, 232; F. S. Odell, 233.

SEWERS AND SEWERAGE.

"Separate Sewer System Without Automatic Flush Tanks." (With Discussion.) F. S. Odell, 223.

SKINNER, F. W. High Masonry Dams, 505.

SMITH, J. Waldo. "Experiments on the Discharge of a 30-In. Stop-Valve," 235, 246.

SNOW, J. P. Life of an Iron Railroad Bridge, 325.

SPECIFICATIONS.

Arch Bridges over Streets Between Callowhill and Arch Streets, on the Line of the Philadelphia and Reading Terminal Railroad. Joseph M. Wilson, 171.

Excavation and Masonry on the Philadelphia and Reading Terminal Railroad Company's Lines in the City of Philadelphia. Joseph M. Wilson, 166.

Furnishing, Manufacture and Erection of Structural Iron Work for the Train Sheds, etc., of the Passenger Station of the Philadelphia and Reading Terminal Railroad Company at Twelfth and Market Streets, Philadelphia. Joseph M. Wilson, 175.

STANFORD, H. R. "Notes on Manufacture and Properties of Malleable Cast-Iron," 1.

STANWOOD, JAMES H. Life of an Iron Railroad Bridge, 310.

STARLING, WILLIAM. "The Discharge of the Mississippi River," 347.

STEARNS, F. P. The Temperature of Lakes, 112.

STEEL.

"The Physical Qualities of Acid Open-Hearth— as Compared with Carbon— of Similar Tensile Strength." H. H. Campbell, 285.

SURVEYING. STADIA.

"Topography on the Survey of the Mexico-United States Boundary." (With Discussion.) J. L. Van Ornum, 259.

SYRACUSE, N. Y.

"The Water-Works of—" William R. Hill, 23.

"TEMPERATURE OF LAKES." Desmond FitzGerald, 67.

Discussion: J. T. Fanning, 110; C. G. Darrach, 110, 180; F. P. Stearns, 112.

"TESTS OF FIRE-PROOF FLOORING MATERIAL." George Hill, 542.

THOMSON, JOHN. Consumption and Waste of Water, 205.

TIMBER.

Strength of— "What is the Life of an Iron Railroad Bridge?" (With Discussion.) 301, 313, 320, 323, 327.

"TOPOGRAPHY ON THE SURVEY OF THE MEXICO-UNITED States Boundary." J. L. Van Ornum, 259.

Discussion: F. V. Abbot, 279; W. L. Webb, 279.

Correspondence: Kenneth Allen, 280; William B. Landreth, 281; J. B. Johnson, 283; J. L. Van Ornum, 284.

TUSKA, GUSTAVE R. "Construction of Substructure for Lonesome Valley Viaduct, Knoxville, Cumberland Gap and Louisville Railroad." (With Discussion.) 247, 258.

VALVES.

"Experiments on the Discharge of a 30-In. Stop Valve." (With Discussion.) J. Waldo Smith, 243.

VAN BUREN, JOHN D. "Notes on High Masonry Dams." (With Discussion.) 493, 517.

VAN ORNUM, J. L.

"Topography on the Survey of the Mexico-United States Boundary." (With Discussion.) 259, 284.

Water-Works of Syracuse, N. Y., 63.

VIADUCTS.

"Construction of Substructure for Lonesome Valley Viaduct, Knoxville, Cumberland Gap and Louisville Railroad." (With Discussion.) Gustave R. Tuska, 247.

WAGNER, Samuel Tobias. Philadelphia and Reading Terminal Railroad and Station in Philadelphia, 181.

WATER.
"Consumption and Waste of—" (With Discussion.) Dexter Brackett, 185.

WATER MAINS.
"Moving Two 36-In. Water Mains Without Shutting Off the Water." E. C. Moore, 532.

"WATER-WORKS OF SYRACUSE, N. Y." William R. Hill, 23.
Discussion: J. J. R. Croes, 54, 57; E. Sherman Gould, 56; John Bogart, 58; Charles E. Emery, 60; J. N. Greene, 61; L. L. Buck, 62; G. L. Christian, 62.
Correspondence: J. L. Van Ornum, 63; John W. Hill, 63; L. J. Le Conte, 64; William R. Hill, 64.

"WHAT IS THE LIFE OF AN IRON RAILROAD BRIDGE?" J. E. Greiner, 294.
Discussion: George S. Morison, 308: T. C. Clarke, 309; Joseph M. Wilson, 309.
Correspondence: James H. Stanwood, 310; Onward Bates, 312; C. D. Purdon, 314; G. B. Hazlehurst, 315; M. E. Yeatman, 317; L. J. Le Conte, 318; Walter Katté, 318; George H. Pegram, 320; R. Montfort, 323; J. P. Snow, 325; Theod. G. Hoech, 327; C. C. Schneider, 328; J. E. Greiner, 335.

WEBB, W. L. Survey of Mexico-United States Boundary, 279.

WEGMANN, Edward, Jr.
High Masonry Dams, 508.
Moving 36-In. Water Mains, 536.

WILSON, Joseph M.
Life of an Iron Railroad Bridge, 309.
"The Philadelphia and Reading Terminal Railroad and Station in Philadelphia," 115, 184.

WORTHEN, William E. High Masonry Dams, 505.

YEATMAN, M. E. Life of an Iron Railroad Bridge, 317.

NORTHEASTERN UNIVERSITY LIBRARIES
3 9358 00860145 9

NORTHEASTERN UNIVERSITY LIBRARIES
3 9358 00860145 9